EVALUATION OF SULFUR MODIFIED FOAMED ASPHALT FOR LOCAL SOIL STABILIZATION

BY

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Dedication

This thesis is dedicated to my family whose love, encourage me and support took me to where I am right now.

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In the name of Allah, the Beneficent, the Most Merciful. All praises and thanks are due to Allah, the Lord of the world for the successful completion of this research work. May His peace be upon the last messenger, Prophet Muhammad, his family and companions.

I would like to take this chance to thank those whose help made it possible for me to complete this work.

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LIST OF ABBREVIATIONS

AASHTO : American Association of State Highway and Transportation Officials

ANOVA : Analysis of Variance

ASTM : American Society for Testing and Material

CBR : California Bearing Ratio

ER : Expansion Ratio

FA : Foamed Asphalt

HMA : Hot Mix Asphalt

ITS : Indirect Tensile Strength

M_R : Resilient Modulus

OBC : Optimum Binder Content

OMC : Optimum Moisture Content

RLPD : Repeated Load Permanent Deformation

SEA : Sulfur Extended Asphalt

SFA : Sulfur Foam Asphalt

ABSTRACT

Full Name : Farag Ali Salem Balbahaith

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Foamed asphalt technology is an effective and economical soil improvement and stabilization technique. It has increased in use and gained acceptance worldwide mainly because of its improved aggregate penetration, coating capabilities, and handling and compaction characteristics. Sulfur asphalt concrete has been used successfully to build test sections on several highways. Sulfur which is found in abundant quantities in Saudi Arabia as a raw material can be used in asphalt pavement. Sulfur utilization is an economical technique and it has many applications, for example, it may be used to reduce the required asphalt cement up to 30%. It can be used to reduce the binder content of foamed asphalt. The global proportion 30/70 sulfur asphalt is used to formulate foamed asphalt. Saudi Arabia is considered a major producer of sulfur, a by-product of oil and gas production, which is produced at the rate of approximately 6000 tons/day and is expected to increase to 10,000 tons/day. This research aims to explore the possibility of producing foamed asphalt by using a 30/70 sulfur asphalt proportion ratio and to use the produced mixture for the stabilization of local soils. In this research, the local soils including sand, marl, and sabkha were stabilized using foamed sulfur asphalt in addition to regular foamed asphalt. Designed mixes were compared and evaluated for specification requirements. Results obtained from this study were analyzed to evaluate the suitability of utilizing the foamed sulfur asphalt technology in construction. Design

mixes were evaluated for shear strength, angle of internal friction and dynamic resilient modulus at 25 °C and 40°C . Dynamic triaxial test was used to evaluate the resistance of the different materials to rutting.

Results indicated that the modified sulfur foam asphalt increase the stability and indirect tensile strength (ITS) of the investigated soil. In addition, Foam Sulfur Asphalt (SFA) mixtures impart the cohesion of the investigated soil, this means that the investigated soils treated with SFA have more performance in compared to FA mixtures.

Finally, regular foamed asphalt mixes have higher rutting than sulfur foamed asphalt mixes. Modified sulfur foam asphalt mixtures exhibited lower permanent deformation compared to the foamed asphalt mixes.

Arabic abstract

الاسم الكامل: فرج علي سالم بلبحيث

عنوان الرسالة: تقييم رغوة الإسفلت المحسنة بالكبريت لتقويم التربة المحلية

التخصص: هندسة مدنية

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تعتبر تقنية رغوة الأسفلت المستخدمة في تحسين التربة ذات فعالية عالية واقتصادية وذلك لعدد من الأسباب من اهمها تحسين نفاذيتها للمواد وخلطها بالإضافة الى سهولة نقلها و دكها. كذلك يستخدم الكبريت في كثير من الخلطات الاسفلتية لإنشاء الطرق, ويعد من المواد التي يمكن الحصول عليها بكميات وفيرة في المملكة العربية السعودية كمادة خام يمكن استخدامها في الرصف الأسفلتي، كما يعد الكبريت من المواد الاكثر اقتصاديا في العديد من التطبيقات، على سبيل المثال، يمكن استخدامه للحد من كمية الأسفلت المستخدم بنسبة تصل إلى 30 ٪ من كمية الاسفلت، كما يمكن استخدامه لتقليل محتوى رغوة الأسفلت حيث تستخدم نسبة الاسفلت الى الكبريت (70/30) لإنتاج رغوة اسفلتية محسنة. وتعتبر المملكة العربية السعودية من أكبر الدول المنتجة للكبريت كمادة ثانوية من انتاج النفط والغاز ، الذي يتم انتاجه بمعدل يقرب من 6000 طن / يوم ، ويتوقع أن يرتفع إلى 10،000 طن / يوم في المستقبل.

ويهدف هذا البحث الى دراسة امكانية انتاج رغوة الاسفلت المحسن بالكبريت لتثبيت التربة المحلية مثل الرمل ، التربة الجيرية (المارل)، و السبخة بالإضافة إلى رغوة الأسفلت العادية ومن ثم تحليل نتائج هذه الدراسة لتقييم مدى ملاءمة استخدام رغوة الاسفلت المحسن بالكبريت في بناء الطرق، وقد تم تقييم الخلطات الاسفلتية بواسطة عدة اختبارات منها قوة القص، وزاوية الاحتكاك الداخلي ومعامل المرونة الديناميكية عند 25 درجة مئوية و 40 درجة مئوية.

اشارت النتائج بان رغوة الاسفلت المحسنة بالكبريت حسنت خواص التربة المحلية (الرمل ، التربة الجيرية (المارل) والسبخة) حيث وجد ان التربة المحلية المعالجة برغوة الاسفلت المحسن كانت مقاومتها للتخدد افضل من التربة المحسنة بالرغوة الاسفلتية العادية بنسبة 12 %.

CHAPTER 1

INTRODUCTION

1.1 General

Foamed asphalt is a mixture of air, water and hot bitumen. It is produced by injecting a small quantity of water to the hot bitumen in a specially designed expansion chamber with certain air pressure. It expands explosively to about fifteen times its original volume. Foamed asphalt (FA) has been successfully implemented in many roads across the world especially in recycling projects. The use of mixture can potentially conserve fresh aggregates and bitumen, minimize waste, save energy and fuel consumption, and reduce greenhouse gas emission (Widyatmoko et al., 2007).

Foamed asphalt can be used in road construction by 'in-plant (ex-situ)' or 'in-place (in-situ)' technology. Also, it may be used to stabilize deficient sands, gravel, or fine crushed rock by imparting sufficient cohesion and resistance to moisture ingress (Lee, 1981).

When compared with traditional stabilization techniques, FA stabilization offers very significant advantages: it is less responsive to extreme weather conditions than cement or emulsion treated mixes; it allows more time for compaction; mixes stabilized with FA provide superior rutting properties, and cure rapidly thereby allowing earlier use by traffic than cement or emulsion treated mixes (Lee, 1981; Soter International, 1994; Bowering and Martin, 1976).

Saudi Arabia expands over a vast land where there is a scarcity of good quality soils. The geological formations and soil cover vary widely from sedimentary rocks in the east to volcanic rocks in the west. Moreover, sand dominated deserts cover about 50 percent of the Kingdom's area.

Foamed asphalt has many characteristics, it is environment- friendly and epitomizes the asphalt industry's drive towards energy efficiency and economical solutions to build new roads or repair existing roads. This technology was lauded by researchers the world over, although it was developed more than (30) years ago. Because there were no equipment's available at that time to produce or apply the product on a commercial scale, it did not gain much acceptance or implementation after its development (Al-Abdul Wahhab et al., 2007).

Sulfur asphalt has proven its advantage when used to build local roads, especially the reduction of the required asphalt content by as much as 30%. At the same time, foamed asphalt may be used to produce sulfur asphalt blends.

The soil stabilization process is the addition of an additive to a soil to improve the engineering properties of the soil. A stabilized soil is one which has improved load-carrying and durability characteristics through the addition of admixture. The main benefits of the stabilization are to reduce pavement thickness, provision of a construction platform, decrease swell potential, and reduction of the susceptibility to pumping as well as to strength loss due to moisture.

There are several admixtures that can be applied to improve the engineering performance of the local problematic soils, which include cement, lime, emulsified asphalt, cutback asphalt and combined stabilizers. The commonly used additives in soil stabilization are lime, cement and fly ash or any combination of these additives, and asphalt.

1.2 Benefits of Foamed Asphalt

The use of foamed asphalt has many benefits in the field of transportation; the two most important in the current political and economic situation are ease in using foamed asphalt with reclaimed asphalt pavement (RAP) versus hot mix and its economic feasibility. Other benefits include the following (Eller and Olson, 2009):

- It is appropriate to be used with fine soils.
- It improves the engineering properties of the asphalt.
- It can be used to reduce moisture susceptibility.
- It is easy to compact and can be compacted after addition of foamed asphalt and sufficient mixing.
- It has good workability and can remain workable for a relative long period of time.
- It can be used under some adverse weather conditions (light rain or cold weather).
- It has no evaporated volatiles compared with hot mix asphalt (HMA) and cutback emulsions.
- The cost of material transport will be reduced.

- The binder content and water will be less than other types of cold mixing.

1.3 Problem Statement

In Saudi Arabia, there is a scarcity of good quality construction materials. For example, Portland cement is used as a stabilizing material for local soils. It is required to enhance the engineering properties of soils, making the development too costly and sometimes impractical. In addition, stabilization techniques are needed to improve the performance of local soils in order to be used for the construction of base or sub-base layers in the harsh arid desert climate.

In Saudia Arabia, plenty of sulfur is produced as the end product of oil and gas production. Large amounts of sulfur product are not used. In fact, the rate of producing sulfur is approximately 6000 tons/day and it is expected to increase to 10,000 tons/day. As a result, the abundance of sulfur can be used in construction and ways to use it should be explored. Specifically, uses should be economical and environment-friendly. In addition, vast amounts of sulfur should be consumed as a stabilizing material and environmental contamination must be prevented as far as possible.

1.4 Objectives

The main objective of this research is to evaluate foamed sulfur/asphalt to stabilize local soils including sand, marl, and sabkha. The specific objectives of this study are as follows:

1. To investigate the possibility of producing foamed asphalt utilizing 30/70 sulfur/ asphalt ratio and to use the produced asphalt in the stabilization of local soils.

2. Evaluate the produced mix in comparison to those prepared with regular foamed asphalt.

1.5 Research Methodology

In order to achieve the stated objectives, this research is subdivided into seven tasks. Four of which are, literature review, material collection and characterization, foamed asphalt optimization, mix design optimization. The remaining three tasks are evaluation of optimized mixes, data analysis and modeling and thesis report writing.

Task one: Literature Review

A comprehensive literature review is conducted in the areas related to the research topic.

Task two: Material Collection and Characterization

i) Material collection: This involves the collection of materials that are to be used in the mix design, which include dune sand, marl, sabkha, and asphalt.

ii) Characterization of material : this involves the tests to be used to evaluate the properties of the selected material.

Task three: Foamed Asphalt Optimization

Laboratory scale foamed asphalt plant WLB 10 which is available at the Highway Lab is calibrated to produce 30/70 sulfur asphalt foamed asphalt. Firstly, the flow rate of water and sulfur/asphalt for the plant is calibrated, and then the expansion ratio or half-life for the foamed asphalt is determined at different operating temperatures. The temperature that gives the highest half-life was adopted for the research. Produced asphalts were evaluated using relevant ASTM specifications.

Task Four: Mix Design Optimization

Laboratory mix design procedures for foamed sulfur asphalt mix and foamed asphalt are carried out based on the cited literature (Wirtgen, 2004) .

Task Five: Mix Evaluation Procedure

Several tests were conducted to characterize the various mixes and evaluate their expected behavior. These tests include:

i) Marshall Stability and Stability Loss (ASTM D 1559)

ii) The Indirect Tensile Strength (ASTM D 4867)

iii) Resilient Modulus Test (ASTM D 4123)

iv) Static Triaxial Test (ASTM D2850)

v) Dynamic Triaxial Test (AASHTO T-307)

Task six: Data Analysis and Modeling

This task involves the following tasks

i) Data analysis: this task involves the analysis of test results obtained from different tests. It shows the characteristics of modified sulfur foam asphalt mixes and foamed asphalt for the comparison.

ii) Modeling: this task involves the modeling that is carried out throughout the determination of relevant variables such as pavement stress and thickness.

Task seven: Thesis Writing

A comprehensive report consisting of all tasks in this study were written.

CHAPTER 2

LITERATURE REVIEW

2.1 Foamed Asphalt

Foamed asphalt can be defined as the kind of bitumen which looks like bubbles. It is the product of asphalt foaming, and it is also known as foamed bitumen or expanded asphalt. It is produced by injecting a small quantity of water into the hot bitumen, resulting in spontaneous foaming. “The physical properties of the bitumen are momentarily changed when the injected water, on contact with the hot bitumen, is turned into steam which is trapped in thousands of small bitumen bubbles” (Al-Abdul Wahhab and Baig, 2007).

Foamed asphalt was invented in 1956 by Dr. Ladis H. Csanyi, who was a professor at Iowa State University. He realized the possibility of using foamed asphalt as a soil binder. Since then, foamed asphalt technology, which allows lower mixing temperatures, has been used successfully in many countries. The original process consisted of injecting steam into hot bitumen. Dr. Csanyi discovered that, during its metastable life, foamed asphalt could be mixed with a variety of soils to improve their properties and produce a road building material. From that time, the foamed asphalt process experienced only limited application on a global scale, primarily due to the exclusive rights of the patent holders on the foam nozzles.

In 1968, this technology was refined by the Mobil Oil organization, which acquired the patent rights to Csanyi's invention and developed the first expansion chamber that mixed water with asphalt to make foam. The foam was created within an expansion chamber, after which it was dispersed through a series of nozzles, onto the aggregate mass (Te Chiu, 2002). This technology has been successfully employed in Europe, Africa, and the Middle East since the late 1980's and is being increasingly adopted in the U.S., Canada, and Australia as its benefits become widely known (Al-Abdul Wahhab and Baig, 2007). In the last few years, South Africa has been considered as the leader in foamed asphalt area. During the period 1996-2002, there were two research programs undertaken to refine the mix design process (Jenkins, 2000). The first research efforts (1996-2000) combined with field experience resulted in the interim technical guidelines for design and use of foamed asphalt treated materials. The second research phase was initiated in 2002, which involves a comprehensive laboratory assessment and heavy vehicle simulator testing of foamed asphalt treated crushed stone. These tests have contributed to the production of design catalogues and charts that will be used in the future (Verhaeghe et al., 2004).

Bissada (1987) carried out some experimental investigation to study the response of foamed asphalt in the hot environment of Kuwait. He used foamed asphalt to stabilize locally marginal sands for bases or sub-bases of local roads. His study showed that the performance of foamed asphalt mixes depends on sand gradation, moisture content, cement and the addition of limestone powder. Finally, he recommended that the structural response of foamed asphalt-sand mixes is similar to the common hot asphalt-sand mixes used in Kuwait.

In 1982, Australia had placed around 2.9 million square meters of foamed asphalt mixtures, generally as a base or sub-base layer. South Africa, New Zealand, Japan, Germany and other countries had all laid lesser coverage of foamed materials by 1982. Also, USA had produced hundreds of kilometers of surface layer mixtures with foamed asphalt in 1982."Recently, the use of foamed asphalt in cold recycling has gained more acceptances in Europe, South Africa, and Asia"(Al-Abdul Wahhab and Baig, 2007).

Several laboratory research programs were planned and executed to explore the possibility of using of foamed asphalt technology in Saudi Arabia (Asi et al., 2002). These researches aimed at developing and improving the common dune sands and sabkha soil for possible use as base or sub-base materials. Also, several variables were investigated to estimate the relative enhancement of dune sand as well as to allow the development of design procedures for the future use of foamed asphalt technology in the harsh climatic conditions of eastern Saudi Arabia. These researches were employed to validate the effects of foamed asphalt and emulsified asphalt, with and without the addition of Portland cement (0% cement and 2% cement), on the strength characteristics of the treated mixes. The results showed a significant improvement in the performance of dune sand and Sabkha soil-foamed asphalt mixes, as compared to that of the emulsified asphalt mixes. The study recommended that, FA stabilization with the addition of 2% cement is the most effective stabilization procedure to improve the quality of Sabkha soil from both strength and economical points of view (Asi et al., 2002; Al-Abdul Wahhab and Baig, 2007).

Te Chiu et al. (2002) carried out a study on the properties of foamed asphalt treated mixes to investigate the engineering properties of foamed asphalt treated bases in

Taiwan. In his study, five samples were treated with varying asphalt contents. In order to meet his objective, the laboratory tests were conducted using local materials. The engineering properties including Marshall Stability, indirect tensile strength, fatigue and resilient modulus were obtained in both foamed asphalt treated and hot recycled mixes. It showed the resilient modulus and fatigue performance data of both foamed asphalt treated mixes and hot recycled mixes. In some cases, foamed asphalt treated mixes show higher resilient modulus and longer fatigue life than that of hot recycled mixes.

The first time the foamed asphalt application was used in Saudi Arabia was in 1997 in Shaybah road. This road (350 km length) was constructed and placed on marl road (sub-base) and the top layer of marl (200 mm) was mixed with foamed bitumen and cement before being laid onto the remainder of the marl road (Al-Abdul Wahhab et al., 2007).

Al-Abdul Wahhab et al. (2007) carried out a research program in the area of foamed asphalt technology. The research was designed to evaluate the performance of foamed asphalt pavement mixes with conventional aggregate of road bases. In their work, they focused on the investigation and evaluation of the possible use of foamed asphalt technology for local roads by using marginal quality construction materials, marl, and reclaimed asphalt pavement (RAP) materials for local applications. They used two different foamed asphalt mixes in their study, granular base class A and B, sub-base material class B, and reclaimed asphalt pavement (RAP) material. The laboratory tests were carried out to investigate the performance of foamed asphalt. In their study, designed foamed asphalt mixes that include class B sub-base and recycled asphalt pavement (RAP) and aggregate in addition to the virgin materials that include class A aggregate, class B aggregate and class B sub-base were subjected to indirect tensile strength (ITS), California bearing ratio (CBR), resilient modulus (M_R) and static triaxial tests to evaluate their engineering properties. Their study recommended that 2% Portland cement was the best percentage to reduce stability loss, RAP mix has the best behavior, and foamed asphalt can be used successfully to improve the quality of sub-base materials and recycled road materials for road base construction.

Jitareekul and Nicholas (2009) carried out an experimental work on foamed asphalt at the University of Nottingham to investigate the deformation resistance of foamed asphalt bound mixtures normally used as base course materials. This work was aimed at investigating the manner in which foamed asphalt stabilized materials deform when subjected to forms of loading similar to those imposed by traffic. They used repeated load triaxial apparatus and the Nottingham pavement test facility to evaluate the permanent deformation behavior of the materials. The materials used in this study were reclaimed asphalt pavement (RAP), limestone aggregate, and foamed bitumen as a stabilizing agent. Cement was also used as an additive. Foamed asphalt was produced from three penetration grades bitumen, PG50/70, PG70/100, and PG160/220, at a temperature of 180°C and water content of 2% by mass of bitumen. Based on the experimental work of this study, they concluded that the rutting resistance of foamed bitumen mixes was dependent on the mixture proportions and penetration grade of bitumen generating the foam. Foamed asphalt mixes that contain a higher proportion of RAP and a softer binder exhibited greater deformation. Also, the addition of a small percentage of cement to the foamed bitumen bound mixtures significantly enhances the resistance against rutting failure.

In new Zealand, a laboratory study had been done to examine the effects of foamed asphalt contents on the strength and deformation behavior of foamed asphalt mixes using different types of tests, indirect tensile strength (ITS), the monotonic load triaxial (MLT) and the repeat load triaxial (RLT) tests. This study was carried out on a specific granular material containing 1% cement with different bitumen contents. The results showed that an increase in foamed bitumen content up to an "optimum" content, increases the ITS but, at the same time, decreases both the permanent deformation resistance measured in RLT tests and the peak strength in MLT tests (Gonzalez et al., 2009).

Several researchers reported that the foamed asphalt treatment has significantly better performance as compared to other recycled road base materials treatment methods, e.g. asphalt emulsion, Portland cement stabilization.

Ramanujam and Jones (2007) carried out a research study to compare the treatment between foamed asphalt (with lime) and emulsion treatment (with Portland cement).They reported that the foamed asphalt section showed significantly better performance in terms of early exposure to traffic and also better rain resistance before applying the wearing course.

Jenkins et al. (2000) also, recommended that the foamed asphalt strategy is often preferred because the asphalt emulsion treatment introduces extra moisture (the continuous phase in the emulsion) into the mix and requires considerably longer curing periods before the road can be opened to traffic (Khosravifar Sadaf ,2012).

Gonzalez Alvaro (2009) carried out a comprehensive research program to study the effects of foamed bitumen on the deformational behavior and performance of pavement materials. The material used in his research were a blend of coarse aggregate with a crushed dust from the Canterbury region and bitumen of an 80/100 bitumen grade. The tests performed were Indirect Tensile Strength (ITS), Indirect Tensile Resilient Modulus (ITM), Repeat Load Triaxial compression (RLT) and Monotonic Load Triaxial compression (MLT). Finally he concluded that the highest ITS was measured in the section with 2.8% foamed bitumen content and 1% cement, and the ITS in the section without cement and foamed asphalt only was about 4-5 times lower than the ITS measured in specimens with cement. RLT specimens without cement performed poorly in comparison with the specimens with 1% cement. The specimens with 1% cement showed higher permanent deformation with increase in the foamed bitumen content, supporting the results from the previous laboratory study.

In summary, foamed asphalt stabilized material provides a potentially fast, cost-effective and environmentally friendly flexible pavement rehabilitation strategy if designed and produced effectively.

2.1.1 Mechanism of Foamed Asphalt

Foamed asphalt is done by heating the asphalt in a tank to a certain temperature (150°C or above), then it is sprayed from the nozzle into the expansion chamber under certain pressure, while the cold water and compressed air are also sprayed from the other nozzles into the same expansion chamber under certain pressure to increase the bubbles. So they have contact with each other uniformly in the expansion chamber, the asphalt exchanges energy with the surface of water droplet heating the droplet to steam, which results in explosive expansion of asphalt, 15 to 20 times volume expansion and a sharp decline of the viscosity. All of these benefit the workability of foamed asphalt with the mix. Figure 2.1 illustrates the mechanism of foamed asphalt.

Foamed asphalt is characterized by two typical parameters which are expansion ratio and half-life. The expansion ratio, a measure of the viscosity of the foam and determines how well is the bitumen, will disperse in the mix. It is calculated as the ratio of the maximum volume of foam relative to the original volume of bitumen. Half-life is the time taken for the foamed asphalt to stay to half of the maximum volume attained (Maurizio, 2003). These parameters are very important in foamed asphalt during its production. The water is very important in the process; when the amount of water is injected into the expansion chamber, the expansion ratio should be increased with subsidence or decay, which is a shorter half-life, as shown in Figure 2.2.

The water application rate and bitumen temperature are the most important factors influencing foam quality. A higher bitumen temperature usually creates better foam.

A sensitivity analysis in the laboratory is recommended to identify a target bitumen temperature for foaming. As with HMA production, temperature limits should be implemented to prevent damage to the bitumen .The best foam is generally considered to be the one that optimizes both the expansion ratio and half-life.

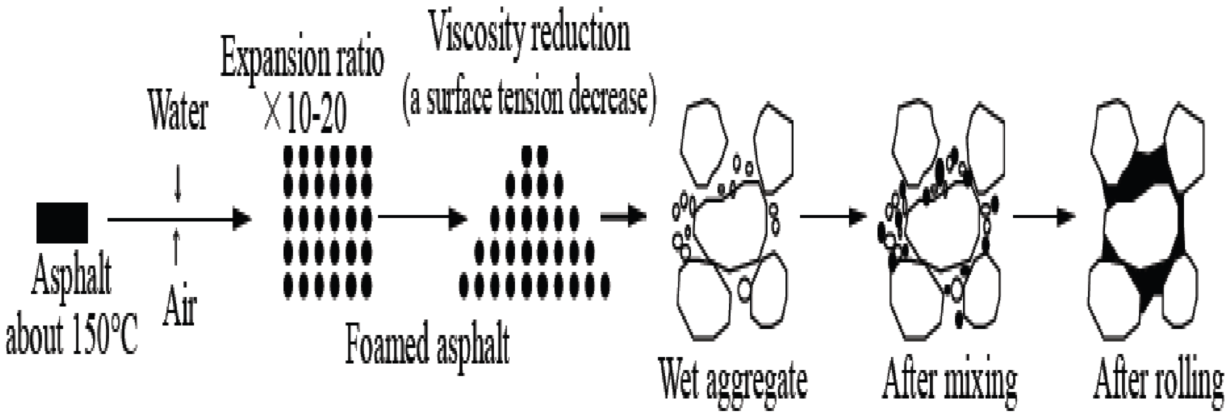


Figure 2.1:The mechanism of foamed asphalt (Wirtgen Gmbh, 2009).

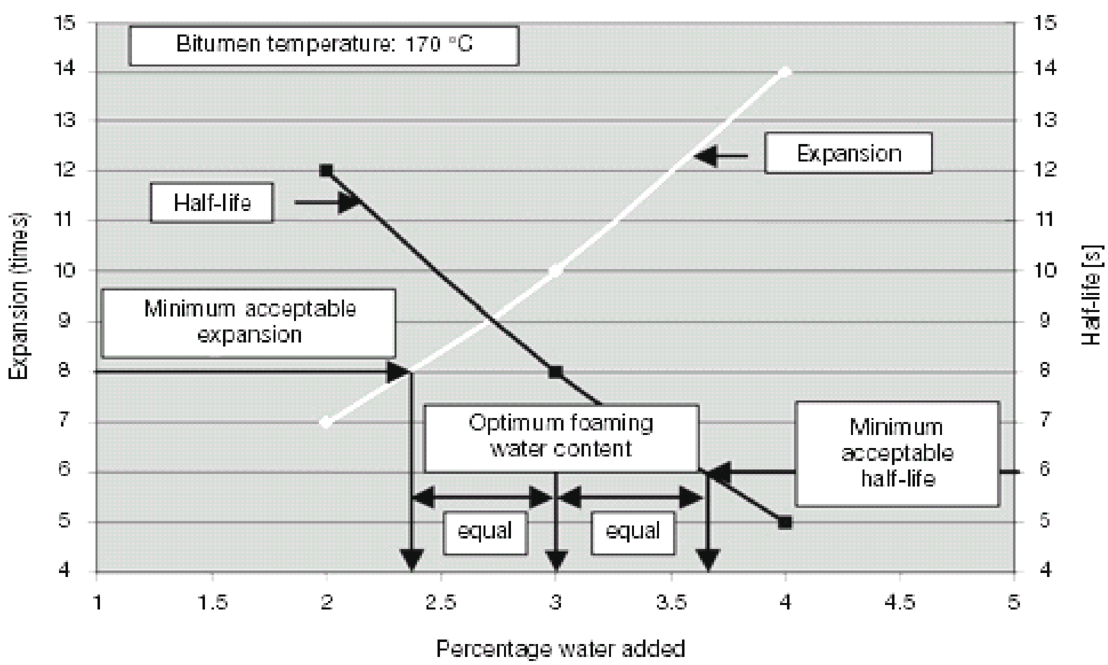


Figure 2.2: Determination of optimum foaming water content (Wirtgen, 2004).

2.1.2 Foamed Asphalt Cold In-Plant Recycling Process

Cold in-place recycling (CIR) is defined as a process of rehabilitating existing asphalt pavement. CIR can be used to repair or remove all types of cracks, keep clearances, minimize the need for new materials, and strengthen pavement. Asphalt is used widely as a stabilizer for sand, weathered sand, gravel, and reclaimed asphalt pavement (RAP). Recently, Foamed asphalt is used as stabilizer material in the recovery of RAP material. In general there are two ways of recovery of RAP material. The first one is referred as in-place cold recycling and the second is referred as in-plant cold recycling.

The disadvantage of in-place cold recycling technology is not controlling the road materials. On the other hand, the in-plant cold recycling technology can improve the quality of material by blending to satisfy the quality requirements of mixture, so that the quality of the mixture can be guaranteed. Because the positive advantage of foamed asphalt such as the good workability with various kinds of aggregates and the viscosity of the asphalt; the asphalt can be mixed easily with a wide range of aggregate (Anlin, 2009).

2.1.3 Benefits of Cold in-Place Recycling with Foamed Asphalt

Actually, there are many benefits of CIR in the field application of transportation. These benefits include the following (Wirtgen GmbH, 2004):

1- Environmental factors. The existing material can be used completely and the volume of new material that has to be imported from quarries is minimized.

2- Quality of the recycled layer. The high quality of mixing in-situ material with water and stabilizing agents can be achieved.

3- Disturbance of sub grade is minimized as compared to pavement rehabilitation.

4- Shorter construction times reduce project costs as well as provide a largely intangible benefit for the road user in the reduced time that the traffic is disrupted.

5- Safety. One of the most important benefits of this process is the high level of traffic safety that can be achieved.

6- Cost effectiveness. The above benefits all combine to make cold recycling the most attractive process for pavement rehabilitation in terms of cost effectiveness.

2.2 Sulfur Extended Asphalt

The availability of sulfur has grown in many countries of the world. Sulfur extended asphalt (SEA) becomes a new technology used for paving roads. It is used by putting a partial amount of sulfur with asphalt. Global utilization of 30/70 sulfur/asphalt mix in road construction and maintenance could consume a significant amount of sulfur annually, making the utilization of sulfur in road construction as one of the primary outlets for sulfur.

SEA was developed by Canada Gulf and has some practical applications. The availability of sulfur in Saudi Arabia has increased due to the increase in number of petroleum and gas refineries (Al-Methael et al., 2007).

In 1978, King Fahd University of Petroleum and Minerals (KFUPM) carried out a field study on asphalt sulfur development. Three SEA test roads were laid in the Eastern Province in cooperation with Gulf Canada and the Ministry of Transport [formerly Ministry of Communications (MOC)]. The study concentrated on three roads by using different percentages of sulfur asphalt. A sulfur/asphalt ratio of 30/70 by weight was used in Test Road 1 (Kuwait Diversion) and Test Road 3 (KFUPM), whereas a higher percentage of 45/55 was used in Test Road 2 (Abu-Hadriyah Expressway). This study recommended that the 30/70 sulfur asphalt is the best rut resistant. In this research, sulfur/asphalt ratio of 30/70 was used as recommended from the previous research to create foamed sulfur.

Al-Abdul Wahhab (2007) carried out a study on the effect of sulfur and sulfur-extended asphalt modifier (SEAM) as compared to conventional local asphalt mixes and verified their adequacies for safe use locally. Throughout the research, it was found that the stability result obtained with sulfur 30/70 mix is higher than the value for the same type of mix with 1% cement added (Al-Abdul Wahhab and Baig, 2007).

Al-Methael and Al-Abdul Wahhab (2010) investigated the possibility of utilizing sulfur as a partial alternative for asphalt in road construction. This study showed that sulfur asphalt has the same workability of conventional asphalt and the emission of fumes associated to sulfur asphalt is lower than the allowable limits with mixing temperature of sulfur asphalt less than 145°C. Also, the study indicated that sulfur can replace 30% of asphalt and this percentage is considered as the optimum replacement.

2.3 Soil Stabilization

2.3.1 Dune Sand

Soils used in pavements may be stabilized or modified through the addition of chemicals or bitumen. The principal benefits of stabilization include a reduction in pavement thickness, provision of a construction platform, decreased swell potential, and reduction of the susceptibility to pumping as well as the susceptibility to strength loss due to moisture.

Most suitable soils for bituminous admixtures are sandy gravels, sands, clayey and silty sands and fine-crushed rock. Highly plastic clays can be treated successfully but may require high quantities of bitumen. The performance and properties of bituminous-stabilized silt-clay soils are affected by clay type, type of exchangeable cations present in clay, soil organic matter and bitumen type and composition. Bituminous uses, applicability, testing procedures, construction and characteristics of the mixture have been discussed in many standards and publications such as ASTM and Asphalt Institute.

Bituminous materials have been used in the United States since 1870. Soil and sand-asphalt stabilization projects were constructed in the united states in 1930 (Terrel et al., 1984). Since then, many low traffic roads have utilized mixed-in-place asphalt stabilization. In addition, hot, central plant asphalt stabilization has been used. Asphalts most commonly used are refined from petroleum. Asphalt cement, cutback asphalts and emulsified asphalts have been used in soil stabilization. Asphalt consists of inert mineral particles impregnated or cemented by bitumen. In general, bitumen is taken to include both tar and asphalt and the use of such material is collectively called bituminous stabilization. Depending on the granulometric composition and the physical properties of the soil, there are four types of bitumen stabilized products: soil bitumen, sand bitumen, waterproofed granular stabilization and oiled earth (Hausmann, 1999).

Al-Abdul Wahhab et al.(1987) carried out a laboratory study to evaluate the feasibility of using blends of dune sand-crusher fines that were stabilized with CSS-1h emulsified asphalt in the construction of low-volume roads in the Kingdom of Saudi Arabia. Several tests were performed on cured Marshall samples such as Marshall Stability, split tensile strength, resilient modulus, fatigue life, and rutting. He used two different percentages of Portland cement .The results indicated that the stability, resilient modulus, fatigue, and rutting characteristics of such mixes were improved significantly. Thickness design charts were developed for the designed mixes, which proved to be suitable for use in hot, arid areas (Al-Abdul Wahhab et al., 1987).

2.3.2 Marl Soil

The Eastern province of Saudi Arabia has unique geotechnical properties and the soils available for construction purposes are mainly marl, dune sand and sabkha. Marl soil is defined as calcareous in nature and it is well known for its heterogeneous nature in terms of composition and properties. Moreover, it is sensitive to changes in water content and it often requires prior treatment without which a significant strength loss will occur upon water flooding (Al-Amoudi, 2010).

Marl is one of the four (i.e., sand, marl, clay and sabkha) predominant types of soil found in eastern Saudi Arabia. Due to the unsuitability of the other three soils, marl soils are uniquely used in the construction of all types of road bases, embankments and foundations. Marl is defined as a soil or rock-like material containing about 35–65% calcareous material as well as varying percentages of clay content (Qahwash AA, 1975, Al-Amoudi, 2010). The term ‘‘marl” is often loosely used to represent, loosely, all types of calcareous materials present in the Eastern Province of Saudi Arabia(Ahmed HR,1995, Al-Amoudi, 2010). Marl, being primarily calcareous in nature, is influenced by the mineral composition, type of parent carbonate mineral present, origin and the formation process, grain-size distribution and degree of cementation. In addition, the variation in density and moisture content, and post depositional changes affect the behavior of this type of soil (Aiban et al., 1995). Consequently, marl generally exhibits a wide variation in terms of its characteristics, engineering properties and definitions.

Marl is abundant in eastern Saudi Arabia in many places such as the Abqaiq, Dhahran, Dammam, Abu Ali, Hofuf, Berri, Fadhli, Jubail, Abu Hadriyah and Safaniyah areas.

The marls in eastern Saudi Arabia, like all marls, vary greatly from one location to another in terms of colour, plasticity, physical and chemical composition and thus engineering properties. Marl colours found in the Eastern Province of Saudi Arabia include, but are not limited to, white, milky, dark and light gray, pink, yellow and brown. Marl plasticity varies from none to moderate, depending on the composition, especially the clay mineral type and content (Aiban et al., 1995).

Marl which is available in abundance in the Eastern Province has poor strength and can only be used as sub-grade layers or as a backfill in base or sub base layers in roads.

Several studies were conducted to stabilize the marl and to improve its engineering properties for use in roads.

Different stabilization admixtures that may have potential application in improving the engineering performance of the local problematic soils include lime, cement, emulsified asphalt, cutback asphalt and combined stabilizer.

Arora and Arabiat (1986) carried out a laboratory study to evaluate the asphalt emulsion treated mixtures for use in base courses for low volume desert in Saudi Arabia. They blended a marl soil with 30% of blown dune sand to meet the criteria recommended by Asphalt Institute (1980) for treating silty sand with asphalt emulsion for road bases. They added a small percentage of Portland cement to enhance the strength and to resist water induced damage. The results showed that emulsion-treated marl-sand mixture are appropriate to be used for low volume roads.

Asi and Al-Abdul Wahhab (1997) carried out a research study to stabilize the marl soil and dune sand by emulsion. They used slow setting emulsified asphalt and medium curing cutback asphalt. Also, they used 2% lime and 4% cement to accelerate the process and reduce the stability. They found that the stabilizing agent had improved the shear strength and resistance of the analyzed soils to water damage. In addition, they concluded that the Portland cement is more effective than lime.

Aiban et al. (1998) carried out a comprehensive laboratory program to assess the performance of cement-stabilized marl mixtures under different exposure conditions. He investigated that the marl used in the construction, similar to other marls, has acute water sensitivity and loss of strength whenever the soil is inundated. In his study program, he used four sections; two of them without additives and the other were treated with 4% cement. The four sections were monitored for four years and he concluded that the cement-treated road sections exhibited superior performance over the untreated ones. Unlike the untreated sections, which experienced various forms of deterioration within a few months after construction, the stabilized sections are still in an excellent condition (Aiban et al., 1998).

Habib ur- Rehman (1995) conducted comprehensive research program into the stabilization of the marl soil in the Eastern Province of Saudi Arabia. His research focused on the improvement and stabilization of marl soil. He investigated both mechanical and chemical soil stabilization techniques and he found that the chemical stabilization using cement was an effective way to improve the inferior properties of these soils.

Al-Amoudi et al. (2010) conducted a laboratory investigation to improve the marl soil to be used as a road base material. He used various tests to characterize and quantify the strength and durability of the studied marl under different field-simulated conditions with and without chemical treatment (lime and cement). He concluded that cement is superior to lime both in terms of strength improvement and durability requirements.

Based on the previous brief literature review on stabilizing marl soil with asphalt, cement and lime throughout the world and in the Arabian Gulf in particular, there is no evidence of stabilizing marl soil with foamed or sulfur foam asphalt. Thus, this research intends to investigate the feasibility of stabilizing marl soil with foamed and sulfur foam asphalt.

2.3.3 Sabkha Soil

Sabkha is an Arabic word to describe recent coastal sediments with a high salt content and which are characterized by very low bearing capacities and low SPT values. It is one of the many types of evaporate regimes that exist around the globe, particularly within tropical zones (Al-Amoudi 1995).Sabkha soils are widely distributed in the Arabian Peninsula. Sabkha soils are not only found in the Middle East but are also worldwide distributed, for example, in India, Australia, USA and Southern Africa where sabkha soils are known by different names (Abduljauwad et al. 1994).In the Arabian Peninsula, sabkha soils are scattered along both the Arabian Gulf and the Red Sea coasts, where some of the major industrial and petrochemical facilities are located. In Saudi Arabia, there are a large number of sabkha soil areas, both coastal and inland especially in the well-populated cities along the Arabian Gulf and Red Sea coasts. Therefore, it is important to search for an expedient technique to stabilize and improve the sabkha properties (Shabel, 2006). There are several papers which have been published about the sabkha characteristics which classified sabkha soil as Muddy Sabkha areas along the sea shore coastal areas and Sandy Sabkha areas inland (Shahin, 2009).

A1-Abdul Wahhab et al. (1994) have conducted a laboratory program about sabkha. They used a sabkha from eastern Saudi Arabia. In his study, two types of liquid asphalts were used, namely emulsion and cutback, at four different percentages. Also, he used lime and cement to improve the strength of the asphalt-stabilized sabkha. The results of this investigation indicate that the sabkha properties (Φ and c) have improved significantly by the addition of cement to the emulsified asphalt. Properties of sabkha have been improved to a lesser extent when lime and emulsified asphalt were conjointly used. Emulsified asphalt at 4 % Portland cement proved to be the best treatment (A1-Abdul Wahhab et al., 1994).

Ahmad (1997) carried out a research program to improve the inferior properties of two eastern Saudi sabkha soils using various stabilizing agents, namely lime, cement, and emulsion, at different percentages for the use in pavements .Unconfined compressive strength, CBR, resilient modulus and durability tests were performed in his research. The results indicated that a cement content of 7% was found to be adequate to effectively stabilize Al-Aziziyah sabkha soil. It met both the strength and durability requirements. He recommended that the stabilized soils can be used as sub-base and sub-grade layers for roads (Ahmad, 1997).

A1-Abdul Wahhab et al. (2006) carried out a research program to assess the performance of Al-Aziziyah sabkha with different stabilizers. He used different stabilizer types with different percentages. The unconfined compressive strength CBR, Clegg impact value was performed in his program. He showed that the cement addition to the soil produced good results, while lime and emulsion could not produce a significant improvement in strength. Also, he concluded that the optimum amount of cement for sabkha stabilization was 7 % that which provided the required strength and durability (A1-Abdul Wahhab et al., 2006).

2.4 Pavement Modeling

2.4.1 Pavement Modeling

Pavement deterioration is a complex process which involves several functional distresses that include pavement rutting, cracking and disintegration with time. The deterioration comes from the interaction between traffic, climate and material over the road lifespan. It controls and determines to a large extent the change in pavement performance over time. Pavement performance is defined as the ability of the road to satisfy the demands of traffic and environment over its design life. So, to be able to capture and project future road condition, one needs specially design tools that will be able to capture these deterioration processes. Performance models are the best approximate predictors of expected conditions (Isa et al., 2005). Also, rutting or permanent deformation is considered as one of the major distress mechanisms in flexible pavements which may occur in all layers of the pavement structure due to weak sub-grade, unstable mix or both.

2.4.2 Permanent Deformation (Rutting) of HMA Mixes

Permanent deformation is considered as one of the major distress mechanisms in asphalt pavement in addition to fatigue and thermal cracking. It is also one of the most common pavement distresses due to repetitive traffic loads, which accumulates in small amounts of unrecoverable deformation caused by each load application and appears as longitudinal depression in the wheel paths of the roadways as shown in Figure 2.3.These deformations have negative effects on pavement, such as lower riding comfort for road users and high maintenance costs (Hafeez, 2009; Oscarsson, 2007).

Flexible pavement rutting is categorized into three stages (primary, secondary and tertiary) as recommended by Mechanistic-Empirical Pavement Design Guide (MEPDG). These three stages can be explained in Figure 2.4. The initial stage, referred to as primary stage, represents a good densification of the material and there are increases in the level of rutting associated with a decreasing rate of plastic deformation with volumetric change. In the secondary stage, there is a small rate of rutting associated with a constant rate of rutting with small changes in volume; however, shear deformations increase at an increasing rate of rutting. The tertiary stage has high level of rutting associated with no volumetric changes (Hafeez, 2009 and Jadoun, 2011).

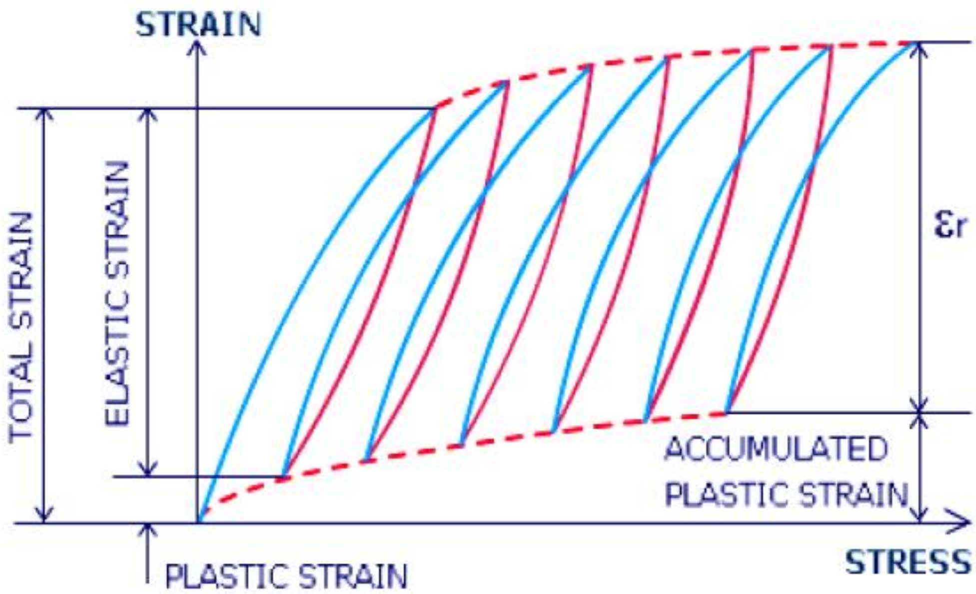


Figure 2.3: Accumulated plastic strains in pavements (Hafeez, 2009).

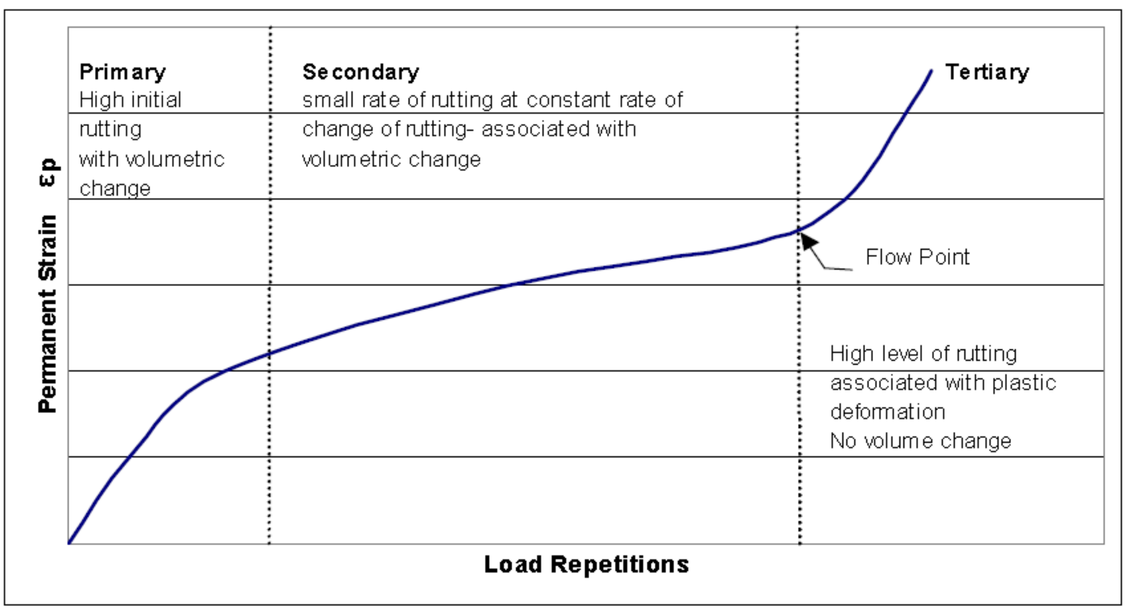


Figure 2.4: Pavement deformation behavior of pavement materials (Ali, 2005).

2.4.3 Permanent Deformation Prediction Models

Several models have been reported in the previous research to predict permanent deformation in flexible pavement. In this research, a few models will be developed.

In 2002, Gareba carried out a research work to investigate the effects of material properties on permanent deformation of asphalt mixtures and methods of its prediction.

The objectives of this study include review and evaluation of available models for permanent deformation of asphalt concrete mixtures, and investigation of the effect of volumetric composition, loading and temperature conditions on the permanent deformation of asphalt concrete in which repeated load triaxial creep and recovery tests were conducted at 250°C and 500°C under varying stress conditions. He concluded that, the elasto-viscoplastic model which is based on strain decomposition approach provides a suitable method for analysis of creep and recovery test results.

Barenberg et al. (1990) recommended the use of the permanent strain accumulation model developed at Ohio State University (Majidzadeh et al., 1981; Hafeez, 2009). This strain model predicts total rutting and considers the rutting rate of the pavement as indicated by the following equation:

$$\varepsilon_p/N = A(N)^m \tag{2.1}$$

Where,

ε_p = permanent strain

N = number of load application

A = experimental constant (depends on material type and stress state)

m = experimental constant (depends on material type).

Majidzadeh et al. (1981) recommended that Equation (2.1) can be used for the development of rutting in pavement layers, asphalt surface and base courses, granular base and sub-base courses, and sub-grade.

Barenberg et al. (1990) revealed that, the model which is related to the log of permanent strain to the log of load repetition appears to be the most suitable and flexible for practical use. This power model is often fixed to the accumulated permanent deformation curve. This model can be plotted as straight line on log-log scale. The slope and intercept of this model can be used as indicators of rutting resistance (Hafeez, 2009).

The model depends on the linear log relationship between permanent strain (ε_p) and the number of load applications (N) (Sugjoon, 2003). It can be mathematically expressed as:

$$\varepsilon_p = aN^b \tag{2.2}$$

Where,

ε_p = cumulative permanent strain

a = intercept from regression

b = slope from regression

N = number of loading repetitions.

The layer elastic theory can be used to measure the progress of rutting with load repetition in which all layers can be modeled using a constitutive model in the form given in Equation (2.3) (partial differentiation form) (Barenberg et al., 1990; Hafeez, 2009).

$$\frac{\partial \varepsilon_p}{\partial N} = \partial \varepsilon_{pn} = \frac{\partial (aN^b)}{\partial N} \tag{2.3}$$

$$\varepsilon_{pn} = abN^{b-1} \tag{2.4}$$

The resilient strain (ε_r) is assumed to be independent of load repetition. The ratio of plastic to resilient strain can thus be defined as:

$$\varepsilon_p/\varepsilon_r = \mu N^{-\alpha} \tag{2.5}$$

The rate of plastic strain (1–b) referred to as permanent deformation coefficient (α) and plastic to elastic strain ratio referred to as a coefficient (μ), can be calculated as follows:

$$\mu = ab/\varepsilon_r \qquad \alpha = 1-b \tag{2.6}$$

Where,

ε_p = permanent strain (rut value)

N = number of load application

a = intercept coefficient

b = slope coefficient

μ = ratio of plastic to elastic response

α = rate of change of the plastic response.

Sullivan (2002) recommended that, α and μ, which are the stress and temperature dependent nonlinear parameters, can be used for modeling the permanent deformation of the mixes.

MEPDG provided a model to predict rutting. This model was developed from the laboratory uniaxial repeated load strain tests and referred to as material model. This model can be used as the basis to estimate the relationships between the predictor variables and the permanent deformation parameters:

$$\varepsilon_p/\varepsilon_r = a_1\, T^{a2}\, N^{a3} \tag{2.7}$$

Where ε_p and ε_r, are the plastic and elastic strains, respectively, at N repetitions of load, and a1 are the nonlinear regression coefficient. This model will be proposed using the triaxial repeated pulse loading (Stephen et al., 2007; Hafeez, 2009).

There are several permanent deformation models. In general, these models can be categorized as layer strain rutting model and shear strain rutting model (Zhou et. al., 2010).

2.4.3.1 Layer Strain Rutting Model

This approach depends on the use of elastic theory and the results of plastic strains are determined by repeated load tests on pavement materials. The approach was initially introduced by Heukelom and Klomp. The plastic strain ε_p is functionally proportional to the elastic state of stress (or strain) and the number of load repetitions was considered. The original concept of this approach was developed by Zhou et al. (2010). The approach can be applied at any material type and at any point of the pavement system. On the other hand, the pavement response of any material must be experimentally determined from laboratory tests for conditions (times, temperature, stress state, moisture, density, etc.) expected to occur in-situ (Sheng Hu et al., 2011).

Obviously, there are three most capable layer strain rutting models, namely Mechanistic-Empirical Pavement Design Guide (MEPDG) rutting model, National Cooperative Highway Research Program (NCHRP 1-40B) rutting model, and VESYS rutting model.

- **MEPDG rutting model**

Mechanistic-Empirical Pavement Design Guide (MEPDG) rutting model depends on the accessible mechanistic-based models and the databases from Long Term Pavement Performance (LTPP) program. It was developed by National Cooperative Highway Research Program (NCHRP 1-37A). The final MEPDG rutting model for HMA is presented below (Muhammet et al., 2010; Zhou et. al., 2010).

$$\varepsilon_p/\varepsilon_r = k1 * 10^{-3.4488} * T^{1.5606} * N^{0.479244} \tag{2.8}$$

Where ε_p is permanent strain, ε_r is resilient strain, T is temperature (ºF), N is number of load repetitions, and k_1 is depth adjustment coefficient and is defined as follows:

$$k_1 = (C_1 + C2 * D) * 0.328196D \tag{2.9}$$

$$C_1 = -\ 0.1039h_{ac2} + 2.4868\ h_{ac2} - 17.342 \tag{2.10}$$

$$C_2 = 0.0172\ h_{ac2} - 1.7331\ h_{ac2} + 27.428 \tag{2.11}$$

Where h_{ac} is total HMA thickness (inch) and D is depth below the surface (inch).

- **NCHRP 1-40B rutting model**

NCHRP 1-40B rutting model is considered as an enhanced version of the MEPDG rutting model. The enhancement was made to adjust permanent deformation constants based on HMA volumetric properties.

$$\frac{\varepsilon_p}{\varepsilon_r} = k_1 (10^{k_{r1}} * T^{k_{r2}} * N^{k_{r3}}) \tag{2.12}$$

Where k_1 is depth adjustment function defined in the MEPDG rutting model. k_{r1}, k_{r2}, and k_{r3} are material properties and defined below.

Constant k_{r1} is defined as follows:

$$k_{r1} = \log [1.5093 \times 10^{-3} \times k_{r1} \times V_a^{\ 0.5213} \times V_{beff}^{\ 1.0057}] - 3.4488 \qquad (2.13)$$

Constant k_{r2} is defined below.

$$k_{r2} = 1.5606 \left(\frac{V_a}{V_{a(design)}} \right)^{0.25} \left(\frac{p_b}{p_{b(opt)}} \right)^{1.25} F_{index} C_{index} \qquad (2.14)$$

Where,

$V_{a(design)}$ = design air voids;

P_b = asphalt content by weight;

$P_{b(opt)}$ = design asphalt content by weight;

F_{index} and C_{index} = fine aggregate and coarse angularity index.

Constant k_{r3} is presented below:

$$k_{r3} = 0.4791 * k_{r3} * (P_b/P_{b(opt)}) \qquad (2.15)$$

Where k_{r3} is slope coefficient; for fine-graded mixes with GI<20, k_{r3} is 0.40; for coarse-graded mixes with 20<GI<40, k_{r3} is 0.70; for coarse-graded mixes with GI > 40, k_{r3} is 0.80; and GI is gradation index (Zhou et al., 2009).

- **VESYS rutting model**

The VESYS rutting model is based on the assumption (or laboratory permanent deformation law) (Sheng Hu et.al., 2011). The model depends on the linear log relationship between permanent strain (ε_p) and number of load applications (N). It can be mathematically expressed as:

$$\frac{\varepsilon_{pn}}{\varepsilon_r} = \left(\frac{ab}{\varepsilon_r}\right) N^{b-1} = \mu N^{-\alpha} \tag{2.16}$$

The rate of plastic strain (1–b) can be defined with a permanent deformation coefficient (α) and plastic to elastic strain ratio can be defined with a coefficient (μ) as shown below:

$$\mu = ab/\varepsilon_r \qquad \alpha = 1-b \tag{2.17}$$

Where,

ε_{pn} = the vertical permanent strain at load repetition, N;

ε_r = peak haversine load strain for a load pulse of duration of 0.1 sec .

μ and α = material properties depending on stress state, temperature, etc.

"MEPDG provided a rutting model developed from laboratory triaxial repeated load strain tests as basis to estimate the relationships between the predictor variables and the permanent deformation parameters" (Zhou et al., 2010; Sugjoon, 2003). Then, the rut depth for any single layer after N load cycles can be written as:

$$R_D = H \times \varepsilon(\mu/1-\alpha) N^{1-\alpha} \tag{2.18}$$

Where H is layer thickness.

There are two rutting models available in VESYS, "Layer Rutting" and "System Rutting" models. The layer rutting model is expressed by:

$$R_D = \int_{N_1}^{N_2} U \, \mu_{sys} \, N^{-\alpha sys} \, dN \tag{2.19}$$

And the system rutting model is expressed by:

$$R_D = \int_{N_1}^{N_2} U_s^{+} \frac{e_t}{e_s} \mu_{sub} N^{-\alpha \, sub} + \sum_{i=1}^{n=1} \int_{N_1}^{N_2} (U_i^{+} - U_i^{-}) \mu_i N^{-\alpha_i} \tag{2.20}$$

Where,

U is pavement surface deflection; U_s^{+} is deflection on top of the subgrade due to single axle load;U_i^{+} and U_i^{-} are deflection on top and bottom of finite layer i due to axle group;e_t and e_s are strain on top of the subgrade due to the axle group and single axle respectively;μ_{sub} and α_{sub} are permanent deformation parameters of the subgrade;α_i and μ_i are permanent deformation parameters of layer i.

The major feature of the VESYS rutting model is to characterize layer properties rather than the global parameters used by the MEPDG. For each layer, the VESYS rutting model requires permanent deformation parameters: μ and α_i.

2.4.3.2 WesTrack Shearing Strain Rutting Model

This is an alternative approach that has been recently proposed, which depends on shear deformations and WesTrack test sections. This approach used a multi-layered elastic system with the asphalt concrete modulus and depends on the repeated simple shear test at constant height (RSST-CH) laboratory test to predict the pavement model. In simple loading, the permanent shear strain in the AC is assumed to accumulate according to the following expression:

$$\gamma i = a \times \exp(b\tau) \times \gamma e \times n\ c \tag{2.21}$$

where τ is shear stress determined at this depth using elastic analysis; γe is the corresponding elastic shear strain; n is the number of axle load repetitions; and a, b, c are regression coefficients obtained from field data, RSST-CH laboratory test data, and the elastic simulations. The asphalt concrete layer rutting due to shear deformation can be determined using the following equation:

$$RD_{AC} = K * \gamma i\ j \tag{2.22}$$

The value of K is 5.5 for a 150 mm (6 inch) layer, whereas the rut depth (RD) is expressed in inches (Zhou et al., 2009; Sheng Hu et al., 2011).

2.4.4 Permanent Deformation (Rutting) of HMA Mixes

The major distresses of flexible pavement are coming from permanent deformation of each layer. As result, thin flexible pavements have to be constructed because permanent deformations are primarily generated from the underlying materials. The permanent deformation models of asphalt layer are relatively well-developed. In addition, the permanent deformation models for underlying layers are not as developed (Zhou et. al., 2009). Several research works emphasize the existing permanent deformation models for underlying materials constructed under thin AC layers.

There are several models that have been proposed for predicting permanent strains. Table 2.1 summarizes the models that can be used to predict permanent strains in materials underlying the pavement surface (Zhou et al., 2009; Sheng Hu et al., 2011).

2.4.5 Rutting Model Selection and Recommendation

From the literature review, the permanent deformation prediction using WesTrack shearing rutting model requires high RSST-CH to characterize the permanent deformation properties of HMA mixes. The disadvantages of the WesTrack shearing rutting model are high variability of RSST-CH and very limited uses and validation (Zhou et al., 2009).

The VESYS model is different from both the MEPDG and NCHRP 1-40B rutting models. The most important feature of this model is the layer rutting model which characterizes layer properties rather than the global parameters used by MEPDG. The most important parameters of the VESYS model are the material's parameters αi and μi. Its disadvantage also are acquiring these layer properties and running repeated load tests for each layer (Sheng Hu et al., 2011). As noted above, the VESYS layer rutting model has been recommended for predicting stabilized layers rutting.

Table 2.1: Summary of permanent deformation prediction models (Zhou et al., 2009).

Reference	Model
Duncan and Chang (1970)	$$\varepsilon_a = \sigma d / k\sigma_{3n} / \left[1 - \left(\frac{\sigma d R_f}{2(C\cos\Phi + \sigma_3 \sin\Phi}\right)\left(\frac{1}{1-\sin\Phi}\right)\right]$$ ε_a = permanent axial strain $k\sigma_{3n}$ = relationship defining the initial tangent modulus as a function of confining pressure (k and n are constants) R_f = a constant relating compressive strength to an asymptotic stress difference C = cohesion , φ = angle of internal friction , σ_d = deviator stress

Table 2.1: (Continued)

Barksdale (1972)	$\varepsilon_p = a + b \log N$ ε_p = total permanent axial strain N = number of load cycles a and b = constants
Monismith et al (1975)	$\varepsilon_p = IN^s$ ε_p = total permanent axial strain N = number of load cycles I and S = experimentally derived parameters
Kenis (1978)	$\left(\frac{\varepsilon_p(N)}{\varepsilon_r}\right) = \mu N^{-\alpha}$ $\varepsilon_p(N)$ = permanent strain due to single or N^{th} load application ε_r = resilient strain at the 200^{th} repetition N = number of load cycles , μ and α = VESYS parameters
Lentz and Baladi (1981)	$\varepsilon_p = \varepsilon_{0.95S_d} * \mathrm{Ln}\left\{1-\left(\frac{\sigma d}{S_d}\right)\right\}^{-0.15} + \mathrm{Ln}(N)\left(\frac{\sigma d}{S_d}\right) n / \left\{1 - m\left(\frac{\sigma d}{S_d}\right)\right\}$ S_d = static shear strength $\varepsilon_{0.95Sd}$ = static strain at 95 percent of static strength $n = (0.809399+0.003769\ \sigma_3)*10^{-4}$, $m = 0.856355+0.049650 \mathrm{Ln}\ \sigma_3$
Tseng and Lytton (1989)	$\varepsilon_a = \varepsilon_0 \exp\left(-\frac{P}{N}\right)^{\beta}$ ε_0, β, and ρ = three parameter model constants
Ullidtz (1997)	$\varepsilon_p = \alpha\left(\frac{\sigma d}{P_d}\right)^{\beta} * N^{\gamma}$ P_d = atmospheric pressure , α, β, and γ = constants

CHAPTER 3

Methodology

3.1 Introduction

The main objective of this study is to assess the possibility of improving the properties of local soils utilizing foamed and sulfur foam asphalt.

To achieve this objective, three different types of soil, namely sand, non-plastic marl and sabkha from the Eastern Province of Saudi Arabia, were treated with different dosages of the (FA) and (SFA) and evaluated using different tests as shown in Table 3.1.

Material characterization consists of evaluating the engineering properties of pavement component material, i.e. soil material and asphalt. The foamed properties consisting of determination of characteristics of foamed asphalt (expansion ratio and half-life) and the optimum water asphalt content are listed in detail in this chapter. Statistical analysis (analysis of variance) has been conducted to determine the effectiveness of combination. The statistical evaluation was conducted using mini-Tab statistical application software. Figure 3.1 summarizes the different tasks of the research flowchart.

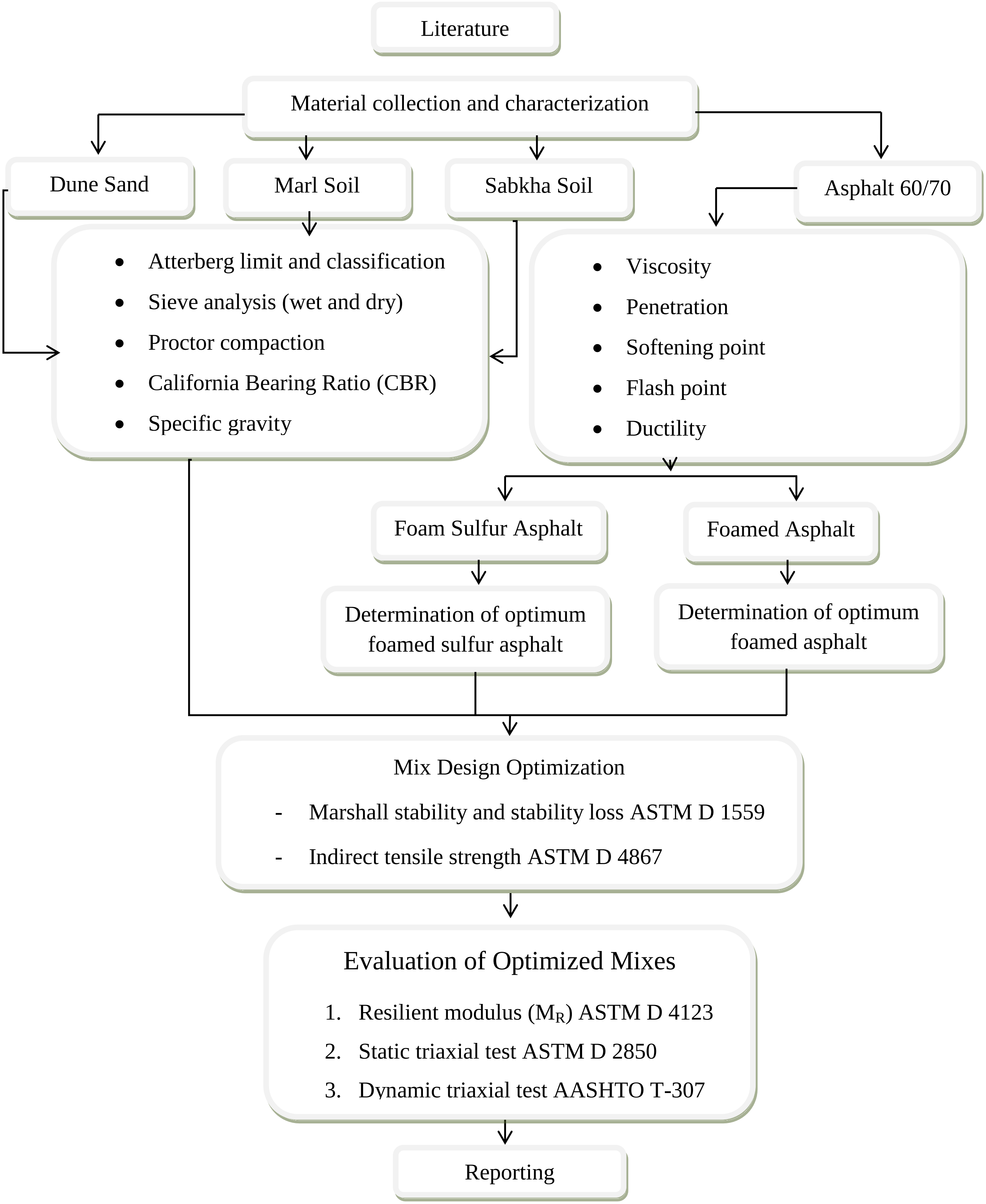


Figure 3.1: Flowchart showing experimental work.

Table 3.1: Evaluated tests for treated soils.

<table>
<tr><th rowspan="3">Soil type</th><th rowspan="3">Test</th><th colspan="8">Asphalt Type</th></tr>
<tr><th colspan="4">Foamed Asphalt</th><th colspan="4">Foamed Sulfur/Asphalt</th></tr>
<tr><th>3%</th><th>6%</th><th>9%</th><th>12%</th><th>3%</th><th>6%</th><th>9%</th><th>12%</th></tr>
<tr><td rowspan="5">Sabkha</td><td>Marshall stability and stability loss</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td>Indirect tensile strength</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td>Resilient modulus</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td>Static triaxial</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td>Dynamic triaxial</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td rowspan="5">Dune Sand</td><td>Marshall stability and stability loss</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td>Indirect tensile strength</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td>Resilient modulus</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td>Static triaxial</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td>Dynamic triaxial</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td rowspan="5">Marl</td><td>Marshall stability and stability loss</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td>Indirect tensile strength</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td>Resilient modulus</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td>Static triaxial</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td>Dynamic triaxial</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
</table>

3.2 Material Selection

The materials used in this research include sabkha, dune sand, marl, asphalt and sulfur. These materials are detailed and characterized separately as follows:

3.2.1 Marl Soil

The marl used in this research was obtained from Dhahran city. This material was collected and taken to the laboratory and sieved, air-dried and disaggregated gently using plastic hammers to pass through an ASTM #4 sieves. The sieved soils were homogenized thoroughly and kept in plastic bags until testing.

These materials were subjected to a series of ASTM tests to characterize the material. These tests include, sieve analysis, atterberg limit, specific gravity, proctor compaction and California Bearing Ratio (CBR).

3.2.2 Dune Sand Soil

The dune sand material used in this research was obtained from Al-Aziziyah which is located 10 km south of Dhahran, Saudi Arabia. The collected material was taken to KFUPM laboratory and stored in bags for testing.

A series of ASTM tests were performed on this soil to identify the physical properties of dune sand that include sieve analysis, atterberg limit, relative density and specific gravity.

3.2.3 Sabkha Soil

The Sabkha soil used in this research were collected from Al-Aziziyah zone, which is located 10 km south of Dhahran, Saudi Arabia. This material were collected, air dried, thoroughly mixed and subjected to basic characterization tests and stored for the use of the research. In order to determine the characteristic of this material, Sabkha soil were subjected further testing as per ASTM standard test method to evaluate other physical properties which are of significant for foamed mixes. The tests include Sieve Analysis, Atterberg limit, grain specific gravity, proctor compaction and California Bearing Ratio (CBR).

3.2.4 Asphalt

Asphalt cement used in this research was obtained from Saudi-Aramco Ras-Tannurah refinery. The grade of the utilized asphalt was 60/70, because this grade was the best grade that widely used in all road projects in the Kingdom.

Several ASTM tests were performed on Asphalt cement 60/70 to evaluate its basic physical properties. These tests included viscosity, penetration, softening point, flash point and ductility.

3.3 Physical Tests

3.3.1 ASTM D 422: Sieve Analysis Test

This is very important test that was performed to determine the grain size distribution using US Bureau Standard sieve (USCS).The test was performed by two types of water, distilled water (as is often done) and Sabkha brine obtained from the beach near to the same test pit from which the Sabkha soil was procured (as recommended by Al-Amoudi and Abduljauwad 1994).

3.3.2 ASTM D 854: Grain Specific Gravity

This test was performed on three representative samples passing ASTM No.4 according to ASTM D 854.The sabkha samples were conducted and yielded 2.61, 2.63, 2.58 and the average was 2.61.

3.3.3 ASTM D 423: Atterberg Limit

Atterberg limit tests are generally performed on local soils passing ASTM N0 .40 sieve. Liquid limit and plastic limit were performed according to ASTM D 423.

3.3.4 ASTM D 1557: Proctor Compaction

Proctor Compaction is used to determine the moisture content and the dry density of a soil. The modified Proctor compaction test (ASTM D 1557) was used to determine the optimum moisture content of the investigated soils.

3.3.5 California Bearing Ratio (CBR)

This test was used to evaluate the material to be used in pavement construction. In this research soaked CBR was performed to evaluate the inundation of local soil. The sabkha sample was subjected to the CBR test in same way as for marl as mentioned above. The soil samples were compacted and put in a water tank for 96 hours according to ASTM D1883 using brine water. The test results indicate that the soaked CBR value of sabkha soil was 10 %.

3.4 Foamed Asphalt Properties

This task includes both regular foam asphalt and modified sulfur foam asphalt (30/70 sulfur/asphalt) properties that should be discussed in detail.

3.4.1 Foamed Asphalt Characteristics

Foamed asphalt was produced using laboratory scale WLB 10 plant that is available at KFUPM laboratory as shown in Figure 3.2. This plant consists of a small tank to heat the asphalt and calibrated systems for asphalt, water, and air. It allows predetermined volumes of asphalt, water, and air to be injected into the expansion chamber where the foam is formed and is then discharged through a nozzle.

The major properties that should be taken into consideration during the production of the foamed asphalt are the expansion ratio and half-life. The expansion ratio is a measure of the viscosity of the foam and will determine how well the binder will disperse in the mix, and it is calculated as the ratio of the maximum volume of foam relative to the original volume of the bitumen. The half-life is a measure of the stability of the foam and provides an indication of the rate of collapse of the foam during mixing. It is calculated as the time taken in seconds for the foam to collapse to half of its maximum volume. These two properties can be calibrated by changing the proportion of water that is added to the asphalt and the optimum addition of water determined.

Figure 3.2: laboratory scale foamed asphalt plant WLB10.

The laboratory foaming machine WLB 10 needed to be calibrated before foam production to determine the flow rate of the asphalt at different temperatures and water ratios at a specific pressure (Wirtgen, 2004). The optimum water content was selected to provide the minimum expansion ratio of eight times and minimum half-life of 6 sec as explained in Figure 3.3 (Wirtgen, 2004).

After calibrating the machine, the foam is produced and the required volume of foamed asphalt is discharged directly into a sample of soil, while it is being agitated in a laboratory mixer. Normally, five samples are produced in this way, with varying asphalt contents. Prior to mixing the soil sample with the foamed asphalt, water is added to bring the material to W_{added} of its optimum moisture content for compaction, where water added (W_{added}) is determined as follows (Wirtgen, 2004):

$$W_{added} = 1 + (0.5\ W_{OMC} - W_{air\text{-}dry}) \quad (3.1)$$

where,

W_{added} = pre-mixing water to be added to the sample.

W_{OMC} = optimum moisture content.

$W_{air\text{-}dry}$ = water in air dried sample.

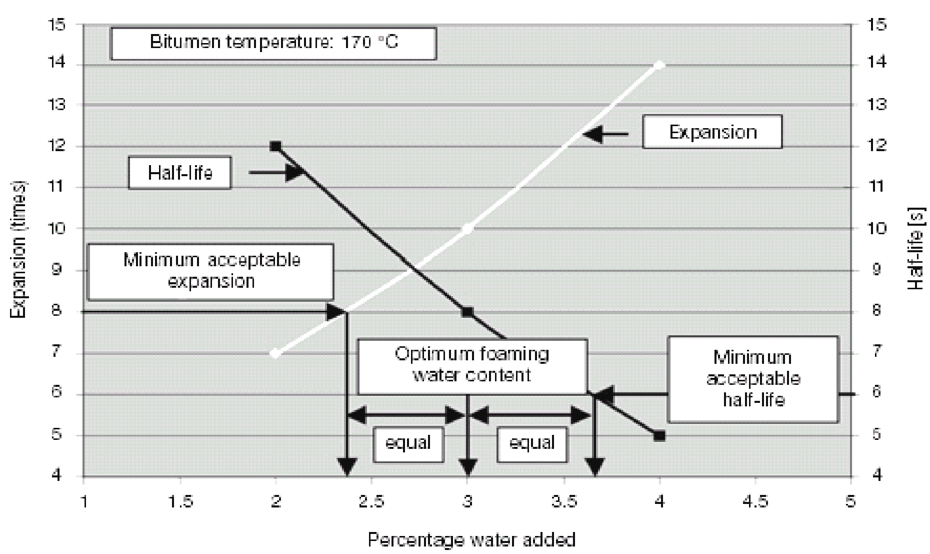


Figure 3.3: Determination of optimum foaming water content (Wirtgen, 2004).

3.4.2 Modified Sulfur Foam Asphalt Characteristics

The sulfur foamed asphalt was produced using the same laboratory scale foamed asphalt WLB 10 that was mentioned above. The laboratory scale foamed asphalt plant shown in Figure 3.1 was used to produce (30/70 sulfur/asphalt) foamed asphalt. The production of sulfur foam asphalt was the same as the production of foamed asphalt. Firstly, the sulfur was heated until melting temperature and then mixed with hot ordinary asphalt before being placed in a foam plant. After that, the mixed sulfur asphalt was put in a foam asphalt plant to produce the foam. Before producing the foam, the flow rate of water and sulfur/asphalt for the plant was calibrated and the expansion ratio and half-life for the modified sulfur foam asphalt were determined at different temperatures (130°C to 150°C) and the water ratio at a specific pressure. The recommended difference between water pressure and air pressure is 1 bar (Wirtgen, 2003). Therefore, the flow gauge was calibrated by measuring the water flow rate at 5.5 bars while the air pressure was set to 4.5 bars.

3.4.3 Mixing with Foamed Asphalt

Before mixing, the WLB10 laboratory foaming was set at the required temperature for foamed and sulfur foam asphalt mentioned above, the foaming water flow was adjusted to the required water flow of foamed and sulfur foam asphalt and the discharge time was adjusted to add the target bitumen content.

The mixer available at the KFUPM Laboratory is a Hobart mixer, and can produce batches of about 10 kg of material for each bitumen discharge. To add the water, the local soil blends were placed in the bowl of the Hobart® mixer and mixed for five minutes to ensure that moisture was uniformly added to the soil sample. The same water content as calculated using Equation 3.1 was added to all the mixes studied to avoid the introduction of another variable into the study (moisture).

A few seconds before the mixing of the soil samples with foamed asphalt and sulfur foam asphalt, a final foam check was conducted to verify that the WLB10 was working properly. The bowl and the mixer were placed under the foaming nozzle of the WLB10. The Hobart mixer was set at fast mixing speed, to ensure a high mixing energy, representative of field conditions. Once the mixer was set working, the foam was discharged and mixed with the soil sample for 30 seconds (Wirtgen, 2004). Also, 2 % of cement was added to the soil sample after it is mixed with the foam to increase the strength of the soil sample.

After completing the mixing, the samples were subjected to Marshall Compactor of 75 blows per face and then put in an oven at 40 $^{\circ}$C for 72 hours curing time prior to testing as shown in Figure 3.4.

Soaked ITS and Marshall stability samples were treated according to the same procedure as used by Jenkins, Rugo and Van deVen (Jenkins et al, 1997). This entails the immersion of samples in a vacuum desiccator's cabinet for an hour at 25°C and at a pressure of 30 mm of mercury. The pressure was then released and the specimens left in the water for another hour, after which they were tested (Weston, 2001).

Figure 3.4: Compacted samples mixed with foamed asphalt.

3.5 Mix Design and Evaluation Procedures

The major properties of concern in stabilized pavements are stability and durability. Stability can be defined as the resistance to deformation and resistance to flow or rutting, while durability refers to the resistance to the effects of weather or its combination with other forces.

In general there are several mix design methods used throughout the world, e.g. Marshalll Mix design method, Hubbard-field mix design method, Hveem mix design method, Asphalt Institute Triaxial method of mix design, super pave, etc. In this research, Marshall mix design will be used and it will be discussed in detail.

This chapter details the mix design of stabilized mixes carried out to optimize and evaluate the engineering properties of stabilized mixes. A series of ASTM tests were performed on stabilized mixes to characterize the various mixes compacted by the

Marshall method as discussed in Chapter 3. These tests were carried out to simulate the behavior of stabilized mixes under field conditions. The tests conducted to optimize the stabilized mixed include:

- Indirect tensile strength ASTM D 1559.
- Marshall stability ASTM D4867.

Furthermore the tests conducted to evaluate the optimized mixes include

- Resilient modulus test ASTM D
- Static triaxial test ASTM D 2850.
- Dynamic triaxial test ASHTO-307

The experimental program for each test is described and the results are presented along, with some observations.

3.5.1 ASTM D 1559: Marshall Stability Method

The Marshall Stability Test is used to measure the maximum load sustained by the bituminous material at a loading rate of 50.8 mm/minute. This test is easy, practical and economical test. The Marshall test is conducted to measure the stability using a stability machine as shown in Figure 3.5. The specimens are loaded at a rate of 2 inch/min until failure. The maximum load is recorded as the stability value and the deformations undergone by the specimen (.01 inch) during loading is measured and reported as flow value.

Figure 3.5: Marshall stability system.

3.5.2 ASTM D 4867: Indirect Tensile Strength Test

Indirect tensile strength test is a simple and economic test used for stabilized materials. ITS is a conventional and practical test used to estimate the strength of FB mixtures and to select the design binder content as recommended by Lewis [1998].

This test is performed by loading a Marshall specimen with a single load parallel to the vertical diametric plane and performed as per ASTM D 4867. The horizontal deformation at peak load is measured during the test by using LVDT gauges as shown in Figure 3.6. The test is performed at 25°C to determine optimum binder content (OBC) of different mixes. The indirect tensile strength can be calculated as follows:

$$\text{ITS}= \frac{2\,P}{P*h*d} \tag{3.2}$$

where,

ITS = Indirect Tensile Strength [kPa].

P = maximum applied load [KN].

h = average height of the specimen [cm].

d = diameter of the specimen [cm].

Figure 3.6: Indirect tensile strength test.

3.5.3 ASTM D T 2850: Static Triaxial Test

The static triaxial test will be used to determine the shear strength of foamed mixes under confining pressure. The test was carried out on a cylindrical specimen of 100 mm diameter and 200 mm height. Three confining pressures 3.515 kg/cm2 (344.75 kPa), 2.109 kg/cm2 (206.85 kpa **)** and 1.406 kg/cm2 (137.9 kPa) were used for each mix. The samples were loaded axially to failure at the selected constant confining pressure at a strain rate of 1.0 per min. and at a temperature of 25°C. The shear strength of the mix is developed principally from the cohesion (c) of the binder and angle of internal friction (φ) for soil, and it is represented by the general Mohr-Coulomb Equation as follows:

$$\tau = c + \sigma \tan \varphi \tag{3.4}$$

where,

τ = shear strength

C = cohesion intercept

φ = friction angle for the failure envelope

σ = total normal stress on the failure plane.

Figure 3.7 shows the static triaxial machine that was used in this research tests.

The Marshall specimen was fabricated to samples of 5 cm in diameter and 10 cm in height, as shown in Figure 3.8. The mixed samples were prepared at the optimum water and binder asphalt content and compacted to the optimum density using the Proctor compactor. The compactor was calibrated to achieve the required density that achieved by Marshall compactor. This calibration showed that, the mar soil required 90 blows and both sabkha and sand required 80 blows to give the required density. The test method ASTM D 2850 was used to determine the shear strength of the cured samples. The test was performed at three different confined pressures up to failure.

Figure 3.7: Static triaxial test machine.

Figure 3.8: Static triaxial samples.

3.5.4 AASHTO T-307: Resilient Modulus Test

The resilient modulus test is performed using the triaxial test setup by applying a repeated axial load on a soil sample that is mounted inside a triaxial cell. In this research the triaxial dynamic test is used to determine the resilient modulus of compacted mixes in order to AASHTO T 307. In this method, the compacted samples are loaded and different combinations of confining and deviator stresses are applied with a limited number of cycles for each stress condition. During this test, the elastic or recoverable strain (ε_r, see Figure 3.9) is measured, and the elastic or resilient modulus is calculated using the formula:

$$M_R = \frac{\sigma_d}{\varepsilon_r} \tag{3.3}$$

where,

M_R = The resilient modulus.

σ_d = The deviator stress.

ε_r = The recoverable vertical strain.

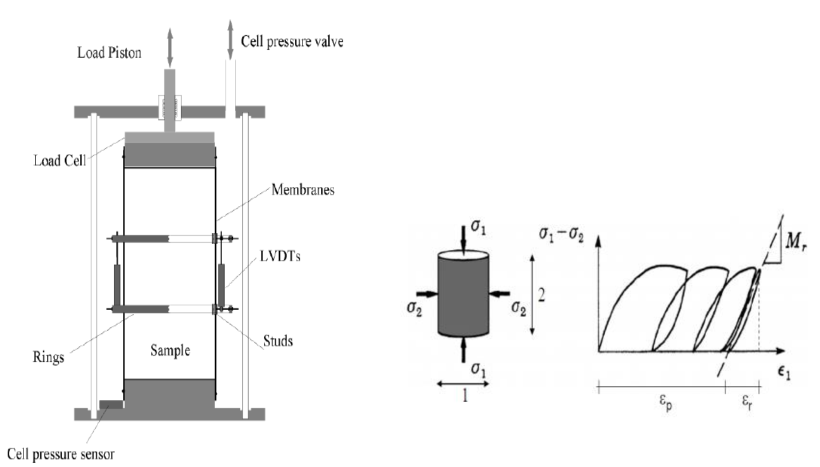


Figure 3.9: Principle of dynamic triaxial.

The mixed samples were fabricated to specimens of 4 inch diameter by 8inch height and compacted using Hveem compactor. The Hveem compactor was calibrated to achieve the required density that was produced by the Marshall compactor. The number of blows per material sample was predetermined. The density of all mixed samples was achieved using three layers of 75 blows. Three specimens were tested for each material at 22°C and 40 °C temperature and different stress levels.

All tests were performed using the Repeated Load Triaxial (RLT) equipment (Figure 3.10) with a closed loop servo pneumatic loading system. Hydraulic air was used as the medium for applying confining pressure.

During testing, the soil specimen in the triaxial chamber was subjected to a repeated cyclic stress and static confining stress conditions. The test starts with a conditioning phase by applying a minimum of 1000 repetitions of load equivalent to a maximum stress of 27.6 kPa and confining pressure of 41.4 kPa for combination (0) as shown in Table 3.2. This point eliminates the effect of the interval between compaction and loading and minimizes the deficient contact between the sample cap and the test specimen.

Different levels of deviator and confining stress were applied to the specimen to measure the resilient modulus according to AASHTO standard. The sequences of stress are summarized in Table 3.2. The load pulse used in this study had a 0.1 sec which corresponds to a 30 mph actual tire speed .The load is applied at a frequency of 60 repetitions per minute and a setting load of about 5 psi was used to hold the specimen in place.

Table 3.2: Level of stress for resilient modulus (ASHTO T-307).

<table>
<tr><th>Combination</th><th>Confining Pressure, Psi</th><th>Deviator stress, psi</th></tr>
<tr><td>0</td><td>15</td><td>15</td></tr>
<tr><td>1</td><td rowspan="3">3</td><td>5</td></tr>
<tr><td>2</td><td>7</td></tr>
<tr><td>3</td><td>9</td></tr>
<tr><td>4</td><td rowspan="3">5</td><td>5</td></tr>
<tr><td>5</td><td>10</td></tr>
<tr><td>6</td><td>15</td></tr>
<tr><td>7</td><td rowspan="4">10</td><td>5</td></tr>
<tr><td>8</td><td>10</td></tr>
<tr><td>9</td><td>20</td></tr>
<tr><td>10</td><td>30</td></tr>
<tr><td>11</td><td rowspan="4">15</td><td>10</td></tr>
<tr><td>12</td><td>20</td></tr>
<tr><td>13</td><td>30</td></tr>
<tr><td>14</td><td>40</td></tr>
<tr><td>15</td><td rowspan="5">20</td><td>10</td></tr>
<tr><td>16</td><td>15</td></tr>
<tr><td>17</td><td>20</td></tr>
<tr><td>18</td><td>30</td></tr>
<tr><td>19</td><td>40</td></tr>
</table>

Figure 3.10: Dynamic triaxial test setup.

3.5.5 AASHTO T-307: Dynamic Triaxial Test

The dynamic triaxial test can be defined as a repeated load test used to estimate the permanent deformation characteristics of foamed bitumen treated materials. The triaxial test setup is created by applying relative stresses and creating strains, as shown previous in Figure 3.9, so that the elastic range of a particular material is not exceeded.

The triaxial dynamic modulus test for the determination of the resilient modulus of soils and aggregate materials was standardized in 1999 as ASHTO T-307. The test consists of applying an axial sinusoidal (haversine) compressive stress to an unconfined cylindrical test specimen.

The triaxial dynamic modulus test was used by many researchers for the determination of dynamic properties of cylindrical soil/asphalt mix specimens. A variable lateral pressure was used and sinusoidal vertical pressure was varied over a range to simulate the nature of loading on the highways or at airports. Triaxial dynamic tests also permit the determination of additional fundamental properties such as Poisson's ratio and the phase angle as functions of the frequency of loading, the number of load cycles and temperature.

3.6 Mix Design Optimization

The local soil include sabkha soil, dun sand and marl soil were mixed with two types of asphalt, regular foam asphalt (FA) and modified sulfur foam asphalt (30/70 sulfur asphalt SFA) to compare the foam-based layer with modified foam sulfur –based layers mixes through their effect on the pavement structure .The mixes treated with foam asphalt were denoted by FA and those treated with modified sulfur foam asphalt by SFA.

The optimum water content for foamed asphalt (FA) and modified sulfur foam asphalt (SFA) was determined and used in all of the soil mixes. The Hobart mixing was used to blend the local soil with FA and SFA separately. Prior to testing, mixed samples were subjected to the Marshall compactor with 75 blows for each side and put in an oven for 72 hours at 40°C. After that, samples were extruded from the mold by an extractor as shown in Figure 3.11 and submerged in water at 25°C for an hour with vacuum and an hour without vacuum.

In order to determine the optimum residual foamed asphalt and residual sulfur foam asphalt for all of these mixes, samples with 3%, 6%, 9% ,12% and 15% of foamed asphalt and modified sulfur foam asphalt (30/70 sulfur asphalt) were tested using indirect tensile strength. After determining the optimum residual foamed and modified sulfur foam asphalt, samples were prepared at optimum residual foamed asphalt to test the stability and to evaluate the optimized mixes. Three dry samples and three soaked (3 replicates for each) were conducted for stability and indirect tensile strength. The resilient modulus, static and dynamic triaxial tests were conducted at the optimum residual foamed and sulfur foam asphalt to evaluate their engineering properties.

Figure 3.11: Sample extractor.

3.7 Dynamic Triaxial Test for Sub Grade

The triaxial fabricated soil sample was 4 in diameter and 8 in height prepared from local soil at optimum moisture content and maximum dry density. The local soil was mixed with OMC for two minute using the Hobart mixer. The marl and sabkha soil were compacted using kneading compactor and the number of layers and number of blows for each soil required to achieve the target dry densities were established using trial-and-error. Both soils were compacted and kept in the mold for 12 hours at room temperature to permit easy removal, after that it was kept in the room for three days curing prior to testing. The Dune sand was compacted using shaking table according to ASTM D 4252.

Subgrade materials were subjected to triaxial test to determine the permanent deformation characteristics of these materials which were mixed, compacted and tested using the triaxial device and procedure mention above.

The compacted soil was tested under 5 psi confining pressure and 10 psi deviator stress since these values are reasonable for simulating the stress received by the sub base materials when loaded by traffic loads (Ramadhan, 1988).

The resilient modulus of subgrade can be calculated by the Equation (South African Council on Scientific and Industrial Research (CSIR))

- M_R for subgrade soil

$$M_R \text{ (psi)} = 3000 * CBR^{0.64} \tag{3.4}$$

CHAPTER 4

RESULTS AND DISCUSSION

4.1 Introduction

This chapter presents the results of the experimental work for the behavior of the soils used in this research (i.e., non-plastic marl, dune sand and sabkha) before and after being stabilized with FA and SFA.

The results obtained were discussed for each test separately as shown below.

4.2 Material Characteristics

The test results of the local soil characteristics were discussed as following.

4.2.1 Grain Specific Gravity

The grain specific gravity of the investigated soil was determined in accordance with ASTM D 854. Three samples of soil were tested to determine the soil specific gravity. Table 4.1 presents a summary for the specific gravity of the investigated soils. The results indicated that the sabkha soil has higher specific gravity than marl and dune sand because of the cementations properties of these materials. Similarly, dune sand has higher specific gravity than marl soil due to high voids.

It can be seen that marl soil has specific gravity of 2.69 which fall in the range of 2.64- 2.92 as reported by (Ahmed, 1995) and dune sand has a specific gravity of 2.64 which falls in the gravity range, 2.62-2.70, reported by (Al-Guniayan, 1998). Similarly, sabkha soil has a specific gravity of 2.71, which is around the value reported by (Al-Amoudi, 1994).

Generally, the specific gravity of the investigated soils falls in the range of Eastern Saudi soils according to the studies mention above.

Table 4.1: Specific Gravity of investigated soils.

Material	Specific gravity
Marl	2.69
Sabkha	2.71
Dune Sand	2.64

4.2.2 Atterberg Limit

Plasticity tests were performed on investigated soils passing ASTM No.40 sieve in accordance with ASTM D 423 to classify the soil. The test results indicated that the soil sample was difficult to roll to the required thread 1/8 in. Also, it was difficult to get the number of blows. As a result the investigated soils were classified as "non-plastic "and nil liquidity.

4.2.3 Sieve Analysis of Marl Soil

The marl soil was subjected to sieve analysis test in accordance with ASTM D 33 to determine the grain size distribution and to classify the soil.

In this research, the samples passing ASTM No. 4 sieve were subjected to dry and wet sieving (ASTM D 422). Figure 4.1 shows the grain size distribution of marl soil. It can be seen that the percent passing sieve ASTM No. 200 is 22 and 28% when the marl samples were sieved using dry and wet methods, respectively. As a result, the soil is classified as SM and A-3 according to the USCS and AASHTO soil systems respectively, based on both dry and wet sieving.

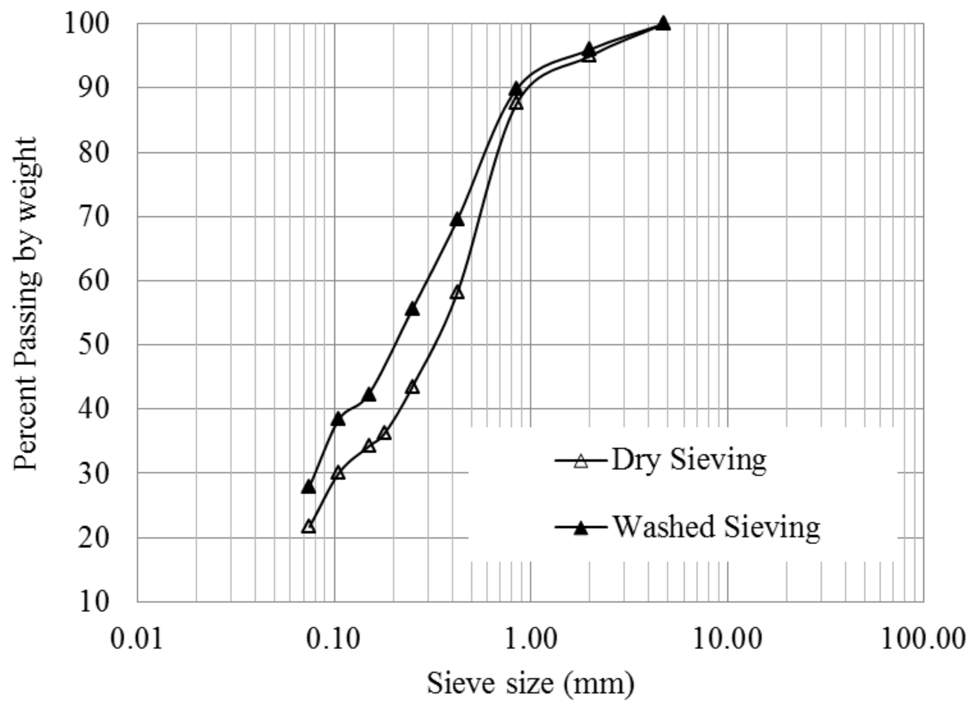


Figure 4.1: Grain size distribution of non-plastic marl.

4.2.4 Sieve Analysis of Sand Soil

The gradation of dune sand is shown in Figure 4.2 and the basic characteristics are listed in Table 4.2. It can be seen that there is no big variation between grain sizes of sand for both the dry and wet sieving which may be because the sand is made of quartz.

Depending on the grain size distribution of sand and atterbirge limit test the soil is classified as SP and A-3 according to the USCS and AASHTO soil systems, respectively, based on both dry and wet sieving.

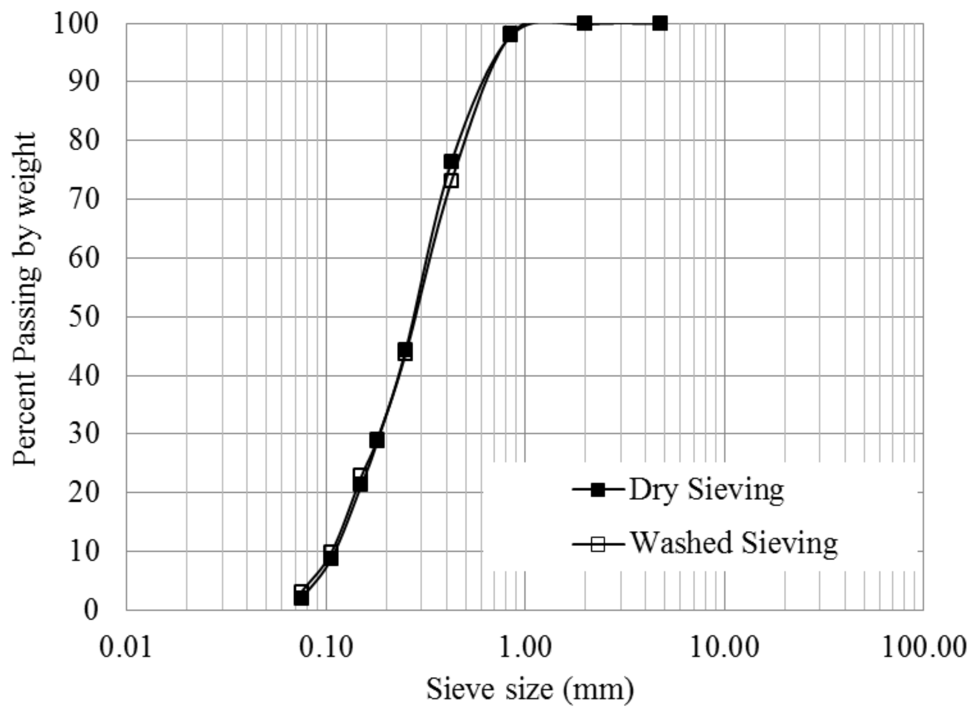


Figure 4.2: Grain size distribution of non-plastic sand.

Table 4.2: Dune sand characteristics.

Physical properties and Test Designation	Value	ASTM limits
Sand Equivalent , ASTM D 2419	79%	-
Specific gravity ASTM C128	2.6	-
Plasticity index ASHTO T-88	Non plastic	-

4.2.5 Sieve Analysis of Sabkha Soil

This is very important test that was performed to determine Sabkha grain size distribution using US Bureau Standard sieve (USCS). The test was performed by two types of water, distilled water (as is often done) and Sabkha brine obtained from the beach near to the same test pit from which the Sabkha soil was procured (as recommended by Al-Amoudi and Abduljauwad 1994). The test results showed that an increase in the percent passing when distilled water was used in the sieving process; however, both curves as shown in Figure 4.3 expose the sandy nature of Al-Aziziyah Sabkha soil. Therefore sabkha is classified as SM and A-3 according to the USCS and AASHTO system for both dry and wet sieving, respectively.

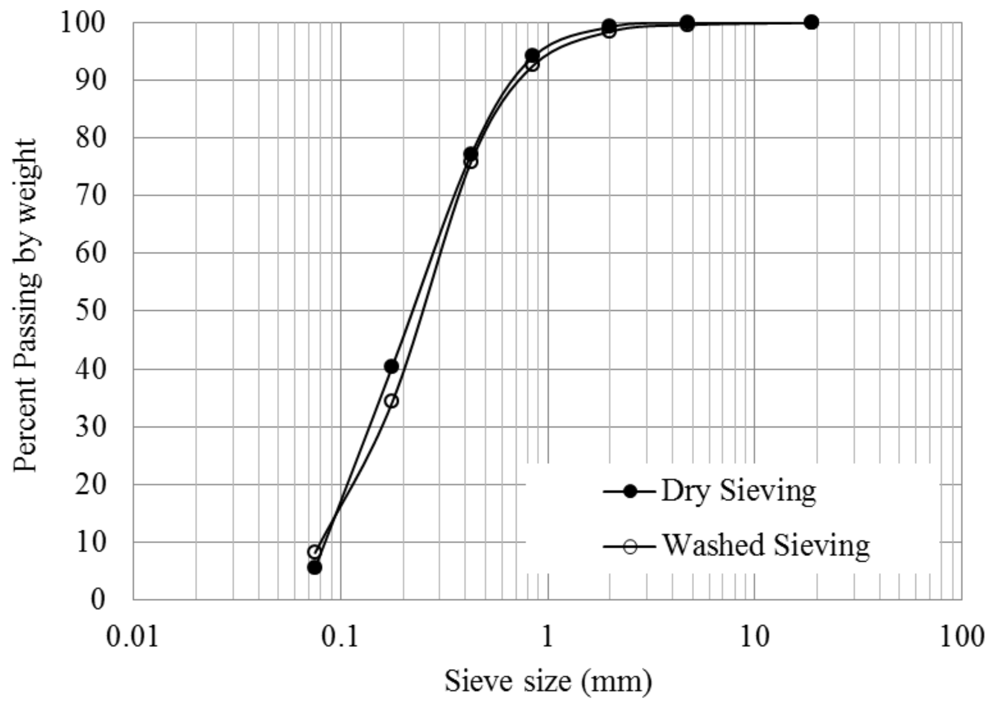


Figure 4.3: Grain size distribution of sabkha.

4.2.6 Proctor Compaction of Marl

Proctor compaction test is used to determine the optimum moisture content at which the maximum dry density of soil is attained. There are two types of compaction tests used in practice standard compaction test (ASTM D 698) and modified compaction test (ASTM D1557) .

The modified proctor compaction test of 18 Ib (8.18 kg) hammer weight and 18 in (45.7 mm) fall height was performed on samples to identify the maximum dry density and the optimum moisture content. The soil samples were compacted in five layers in the CBR mold which has a height of 5 in (127 mm) and a diameter of 6 in (152 mm) with a 25 blows per layer.

The results of these tests indicated that the marl soil optimum moisture content is 13% as shown in Figure 4.4.

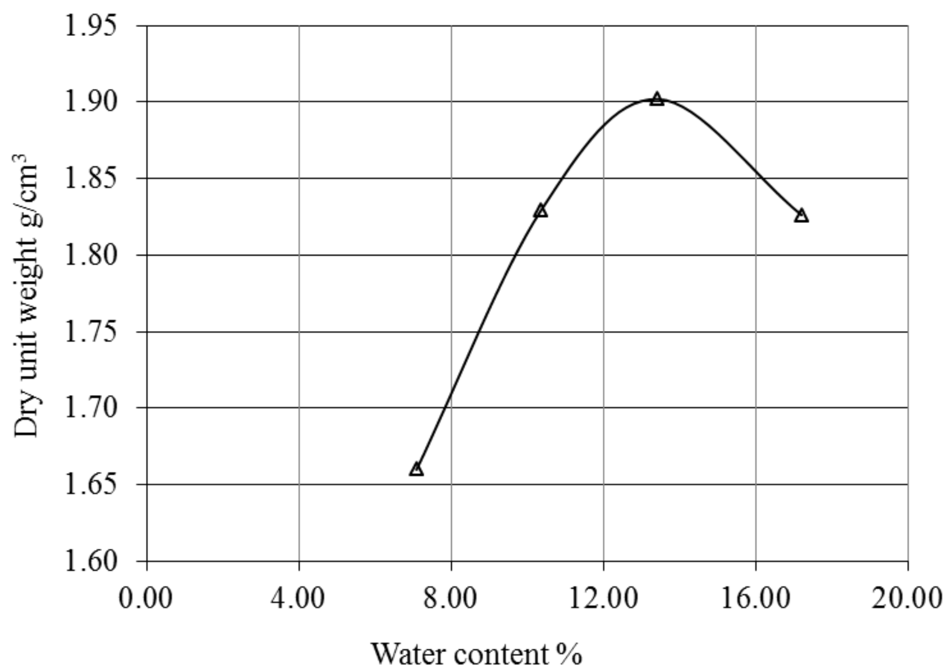


Figure 4.4: Proctor compaction for marl soil.

4.2.7 Proctor Compaction of Sabkha

Proctor Compaction is used to determine the moisture content and the dry density of a soil. The test results as shown in Figure 4.5 indicated that the optimum moisture content of sabkha soil was 12 % that attained dry density of 1.74 g/cm3.

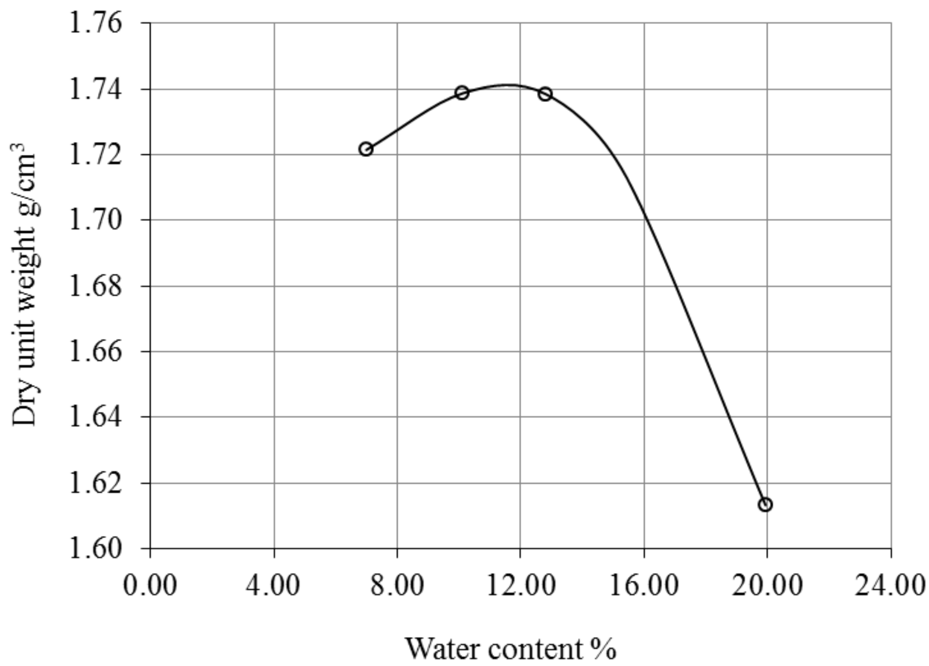


Figure 4.5: Proctor compaction for sabkha Soil.

4.2.8 California Bearing Ratio (CBR) Test

California bearing ratio was developed in California US to assess the possibility of the soil to be used as sub-grade materials in pavements. The test is easy and practical and is used to evaluate the materials to be used in construction.

In this research, marl soil was subjected to soaked CBR test in accordance with ASTM D 1883 to identify the strength behavior of marl soil under the moisture change.

The samples were subjected to CBR test at different moisture content to evaluate the moisture sensitivity of the marl. In this part, soaked CBR test was performed to simulate field conditions in which the soil is flooded in water. After compacting the sample in the CBR mold, they were put in a water tank for 96 hours soaking according to ASTM

D1883 using distilled water. The results of soaked CBR for the investigated soil are presented in Table 4.3.

Table 4.3: CBR value for investigated material.

Material type	Soaked CBR %
Marl	25
Sabkha	10
Dune Sand	15

4.3 Comparison Between Material Properties

This part summarizes the test results of the physical properties of selected material. The main properties of these materials are shown in Table 4.4.

Table 4.4: Material characteristics

Test Type	Marl	Dune Sand	Sabkha
Grain specific gravity	2.57	2.60	2.61
P.I	N.P	N.P	N.P
Optimum water content (%)	13	11	12
Maximum Dry density g/cm3	1.90	1.98	1.74
Soaked CBR %	25	15	10
Classification - AASHTO - USCS	 A-3 SM	 A-3 MS	 A-3 SP

4.4 Asphalt properties

Several ASTM tests were performed on Asphalt cement 60/70 to evaluate its basic physical properties. The results of these tests are listed in Table 4.5, along with ASTM specifications.

Table 4.5: Properties of asphalt cement

Physical properties and test designation	Asphalt utilized	ASTM
Penetration at 25˚C (dmm) (ASTM D 5)	67.6	60-70
Rotational viscosity at 135˚C (centi-poise)	571.75	-
Softening point (˚C) (ASTM D 36)	52.3	49-54
Flash point, Cleveland Open Cup (˚C)	340	223 min
Ductility at 25˚C (ASTM D 113) (cm)	150+	-

The test results indicated that the asphalt has a penetration value of 67.6 dmm, rotational viscosity at 135 ˚C as 571.75 (centi-poise), softening point 52.3°C, flash point 340 ˚C and ductility at 25˚C was 150+.

4.5 Water Content and Foaming Temperature

The asphalt flow rate for foamed asphalt was measured at different temperatures ranging from 160°C to 180°C. It shows that the asphalt flow rate increases with the temperature increase as shown in Figure 4.6. Also, the amount of foaming water was varied at each temperature and the expansion ratio and half-life were measured for each water contents. Figure 4.7 shows the variation of the expansion ratio and half-life at temperature of 180°C. It was found that the expansion ratio and half-life increase with a rise in temperature. By comparing the foam characteristics at the three temperatures, it was found that the asphalt at 180°C produced the best foaming characteristic for foamed asphalt. The optimum water content was found to be 3.5% at 180^{o}C and the water flow rate set to 25 l/h.

On the other hand, the flow rate of water and sulfur/asphalt for the plant was calibrated and the expansion ratio and half-life for the modified sulfur foam asphalt were determined at different temperatures (130°C to 150°C) and the water ratio at a specific pressure. The results show that the temperature 150°C was produced the best sulfur asphalt foaming characteristics and gives the highest half-life as shown in Figure 4.8. The optimum water content was selected according to the criteria shown in Figure 3.6 mentioned above. The modified sulfur foam asphalt water content was 3.6 % and the water flow rate set to 19 l/h.

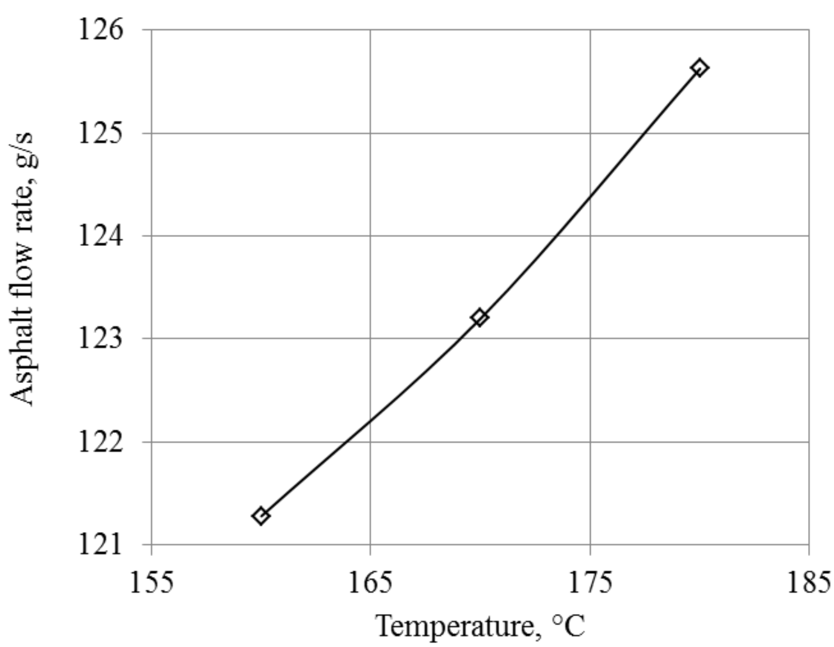


Figure 4.6: Asphalt flow rate variations

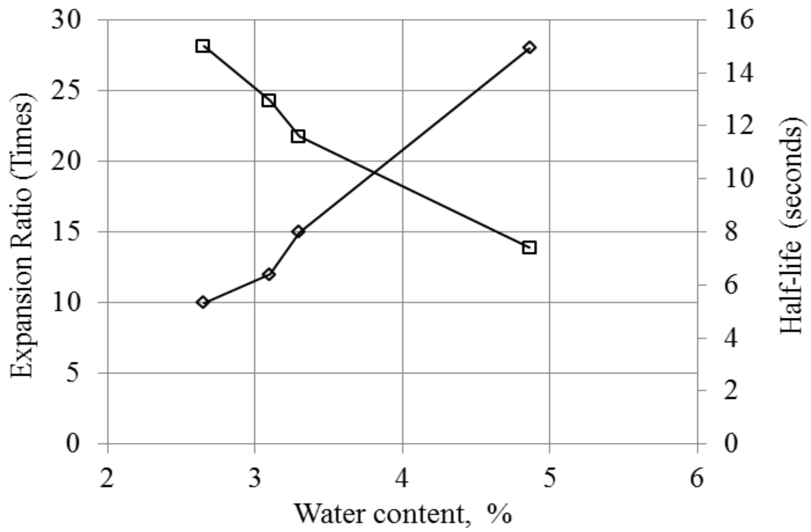


Figure 4.7: Expansion ratio and half-life of foamed asphalt at 180°C.

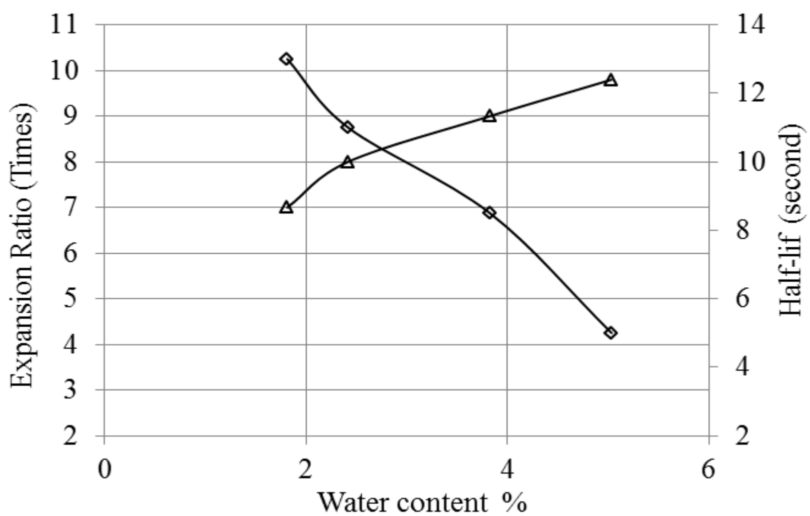


Figure 4.8: Expansion ratio and half-life of modified sulfur foam asphalt at 150 °C.

4.6 Mix Design Optimization

The purpose of the mix design tests is to determine the optimum binder contents of both foamed asphalt and sulfur foam asphalt for compacted mixes. The local soils were mixed with FA and SFA and subjected to the following tests:

The investigated soil was mixed with different percentages of FA and SFA and subjected to indirect tensile strength tests to determine the optimum binder content.

4.6.1 ITS Test Results

The results of split tensile strength are presented in Figures 4.9-4.11 for foamed asphalt and modified sulfur foam asphalt mixes. The test result, indicated that the ITS increases as the bitumen content increases, up to the optimum value, showing that a large volume of bitumen negatively influences the tensile strength of the mix for the soil stabilized. The Australian Road Research Board (ARRB) recommended that the dry and soaked indirect tensile strength should be at least 200 kpa and 100 kpa respectively (SABITA, 1998). The design binder content should be selected as the binder content at which the soaked indirect tensile strength is at the maximum (Al-Abdul Wahhab et al., 2007).

In the case of FA mixes, marl soil has high split tensile strength (535 kpa) followed by sabkha (435 kpa) and dune sand (82 kpa) due to the particle bonding .The maximum soaked ITS occurred when the asphalt contents were 7%, 8 % and 7 % for marl soil, sabkha and dune sand respectively . Similarly, the maximum dry ITS occurred at the same asphalt content.

In the case of SFA mixes, the marl soil exhibited higher ITS (645 kpa) followed by sabkha soil (315 kpa) and dune sand (82 kpa) due to the particle bonding and coating. The maximum soaked ITS occurred when the asphalt contents were 7%, 7% and 8% for marl soil, sabkha and dune sand respectively, as shown in Figures 4.9-4.11.

However, SFA mixes had undoubtedly produced significant improvement in the ITS values for marl soil and dune sand (both soaked and unsoaked) as compared to FA mixes due to the fact that SFA improved the bonding characteristics of these soil particles. On the other hand, sabkha soil treated with SFA exhibited a lower ITS value than when treated with FA due to cementing particles. Furthermore, all mixes satisfy the minimum requirement in both conditions except dune sand as shown in Table 4.6.

Table 4.6: ITS test results for foamed asphalt mixes

Material Type	Test	SFA	FA	Minimum ITS, Kpa
Marl	Foamed asphalt content %	7	7	-
	Dry Split tensile strength , kpa	645	535	200
	Soaked Split tensile strength , kpa	545	430	100
Sabkha	Foamed asphalt content %	7	8	-
	Dry Split tensile strength , kpa	315	435	200
	Soaked Split tensile strength , kpa	228	339	100
Sand	Foamed asphalt content %	8	7	-
	Dry Split tensile strength , kpa	88	82	200
	Soaked Split tensile strength , kpa	61	63	100

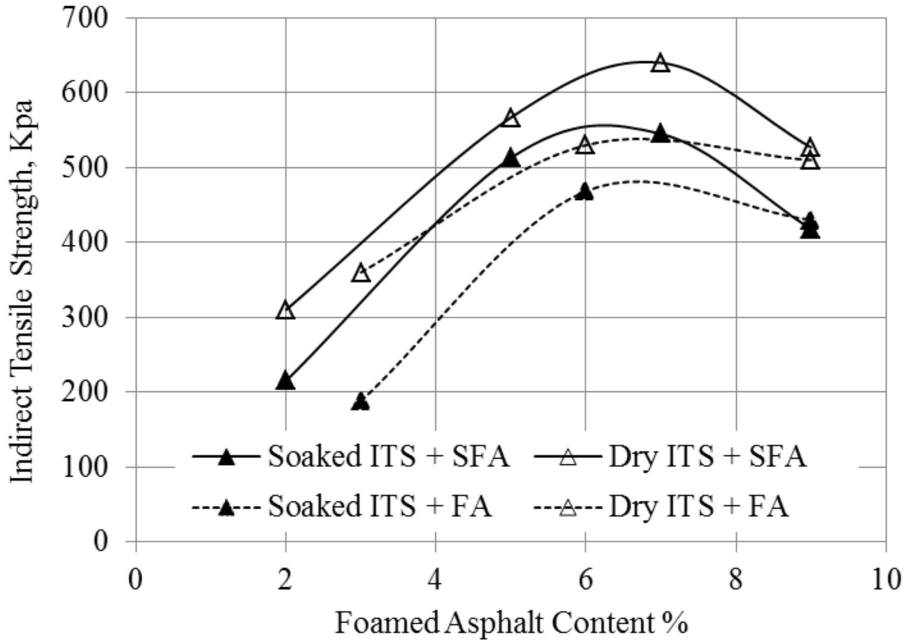


Figure 4.9: ITS for investigated marl soil.

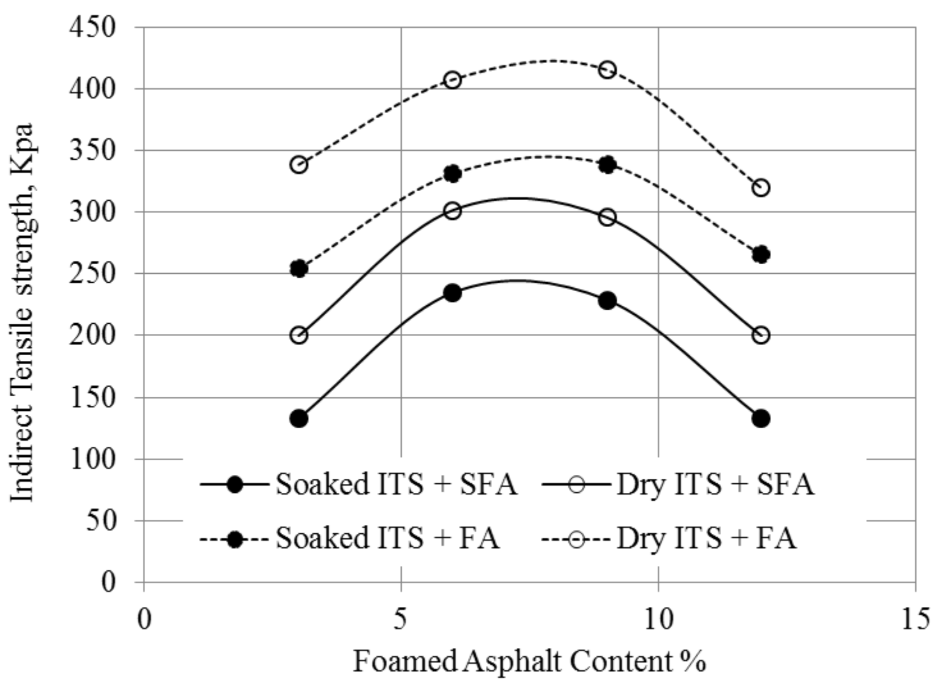


Figure 4.10: ITS for investigated sabkha soil.

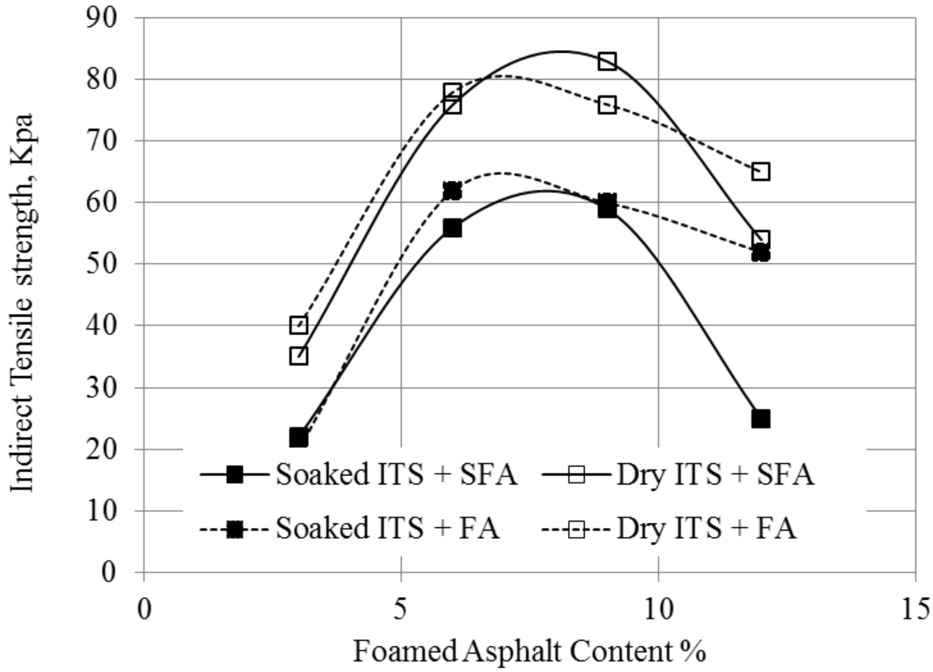


Figure 4.11: ITS for investigated sand soil.

4.6.2 Marshall Stability Test Results

Investigated soil was mixed with FA and SFA at the optimum asphalt content and subjected to the Marshall Stability test in order to determine the stability of the modified mixes and the results are shown in Table 4.7. From the data in Table 4.7, it is clear that the addition of sulfur improved the stability of marl from 28 kn to 34 kn, this was probably due to the good quality of the marl. Also, there is insignificant improvement in the stability of dune sand from 82 kn to 88 kn due to coarse particles. Furthermore, it can be seen that SFA reduced the stability of sabkha (435 kn to 315kn) due to the presence of salts and cementing material.

Table 4.7 summarizes that marl soil treated with FA achieved higher stability (535 kn) followed by sabkha soil and dune sand due to the particle cohesion and small voids. Similarly, marl soil treated with SFA achieved higher stability followed by sabkha soil and dune sand. However, it can be concluded that all the stabilized local soils satisfy the requirements for stability regarding to asphalt institute requirements (6.6 kn) except dune sand in soaked condition. Figure 4.12 shows compacted samples which prepared to Marshall test.

Table 4.7: Marshall stability for foamed and modified sulfur foam asphalt.

Material Type	Test	Modified Sulfur Foam Asphalt	Foamed Asphalt
Marl	Dry stability, kn	34	28
	Soaked stability, kn	19.6	18
Sabkha	Dry stability, kn	20	21
	Soaked stability ,kn	13	8
Sand	Dry stability, kn	7.8	6.8
	Soaked stability, kn	4.8	4.1

Figure 4.12: Marshall samples prepared for stability test.

4.6.3 Comparison Between FA and SFA Mixes

From the results of material mix design optimization, the values for foamed asphalt and modified sulfur foam asphalt mixes of both materials are presented in Table 4.8.

Table 4.8 summarizes the comparison between the foamed asphalt mixes and modified sulfur foam mixes in order of binder content and stability. These results are recommended for use with the soil for further testing .The mixes will be evaluated using dynamic tests for modulus and permanent deformation. From the test results presented in Table 4.8, it can be concluded that the marl soil treated with SFA has the same optimum asphalt content and higher stability and split tensile strength than that treated with FA.

Sabkha soil treated with SFA and FA has the same stability and different optimum asphalt content. The split tensile strength of sabkha treated with FA is higher than that treated with SFA.

Dune sand treated with SFA has slightly improved in stability and tensile strength than when treated with FA. The dune sand has a higher optimum content of SFA than the optimum content of FA (8% - 7%).

Table 4.8: Summary of mix design results.

Material Type	Test	Modified Sulfur Foam Asphalt	Foamed Asphalt
Marl	Optimum Asphalt content%	7	7
	Marshall stability , kn	30	26.7
	Split tensile strength , kpa	645	535
Sabkha	Optimum Asphalt content%	7	8
	Marshall stability , kn	20	20
	Split tensile strength , kpa	315	435
Dune Sand	Optimum Asphalt content%	8	7
	Marshall stability , kn	7.8	6.8
	Split tensile strength , kpa	88	82

4.7 Mix Design Evaluation

This part of the Chapter details the tests that evaluate the optimized mixes of the local soils .These tests include:

4.7.1 Static Triaxial Test

Cured samples were tested using the static triaxial test to determine the shear strength of the stabilized material. The system used to determine the shear strength was static triaxial which mentioned in Chapter 3 (Figure 3.7).

4.7.2 Test Results

The shear strength and related parameters for local soils treated with foamed and modified sulfur foam asphalt mixes are presented in Table 4.9.

The Mohr-Coulomb failure criterion was used to establish the relation between shear strength parameters. The cohesion and angle of internal friction for the treated material were measured and are summarized in Table 4.9.

From the data in Table 4.9, marl soil treated with FA achieved high cohesion value (121 kpa) due to the good bonding of marl particles and fewer voids followed by sabkha soil which achieved cohesion value (104.4 kpa) and very small cohesion value in dune sand (18.8 kpa) due to the small percentage of fines that was not enough to bound soil particles to form conglomerate and increase cohesion. Similarly, in the case of SFA mixes, marl achieved high cohesion value (286.4 kpa) followed by sabkha soil (184.17 kpa) and dune sand (65.17 kpa) depending on the particle bonding .

It can be concluded that, the SFA increased the cohesion of marl soil from 121 kpa to 286.4 kpa due to the good coating and bonding between particles. On the other hand, there is an increase in cohesion value of sabkha soil (104.4 kpa to 184.7 kpa) and dune sand soil from (18.8 kpa to 65.8 kpa) and there was no dramatic change on angle of internal friction .

Generally, SFA mixes increased the strength of the stabilized soils due to the higher viscosity of the modified sulfur foam asphalt binder which improves the cohesion.

Figure 4.13 shows the Mohr coulomb for local soil treated with foamed asphalt and modified sulfur foam asphalt for all treated mixes (marl, sabkha and dune sand) respectively. The general regression model for shear strength which performed a good correlation ($R^2 = 0.93$) is reported as follows:

$$\tau = 14.9 - 0.364\,A + 0.315\,B + 1.0\,C - 0.0636\,\phi \qquad (R^2 = 0.93) \qquad (4.1)$$

where,

τ = shear strength.

A = treatment type (FA or SFA).

B = material type (marl soil, sabkha and dune sand).

C = cohesion.

Φ = angle of internal friction.

Table 4.9: Shear strength parameters for treated soils.

Treatment	Material	τ	C	ϕ
FA	Marl	$\tau = 0.6175x + 121.89$	121.9	22
	Sand	$\tau = 0.5008x + 18.815$	18.8	27
	Sabkha	$\tau = 0.5107x + 104.4$	104.4	27
SFA	Marl	$\tau = 0.4003x + 286.35$	286.4	32
	Sand	$\tau = 0.4849x + 65.865$	65.8	27
	Sabkha	$\tau = 0.5061x + 184.17$	184.17	27

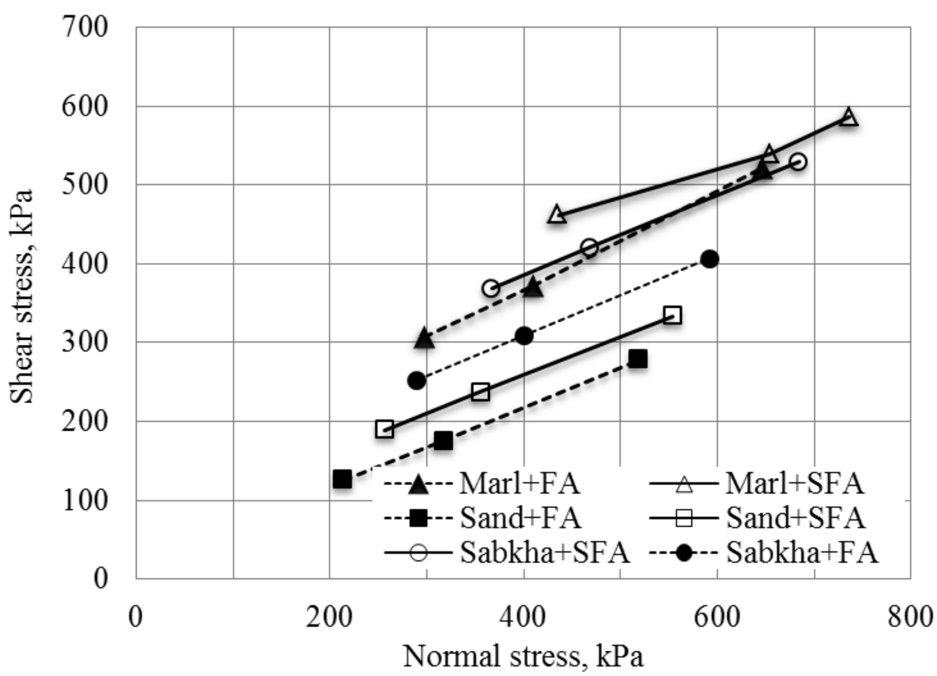


Figure 4.13: Mohr coulomb for all materials treated with FA and SFA.

4.7.3 Resilient Modulus (M_R) Test

Resilient modulus tests were used to test cured specimens at different temperatures and stress levels. The Repeat Load Triaxial (RLT) test is widely used in geomechanics to determine stress-strain behavior of granular materials. The system used to test the specimens was the repeated dynamic triaxial as described in Chapter 3 .

4.7.4 M_R Test Results

A total of 58 resilient modulus tests were performed following the AASHTO T-307 to analyze the effect of the foamed stabilization on the stiffness of three different soils. All triaxial specimens were subjected to resilient modulus tests at different testing temperature 22° C and 40° C, under dynamic loading as described in Chapter 3. This test was done for each specimen prior to permanent deformation.

The results of this test fit a logarithmic relation between the deviator stress and resilient modulus with good regression correlation.

The resilient moduli of investigated soils are presented in Figures 4.14 - 4.15. The results show that the resilient modulus increases with an increase in deviator stress with a small effect of confining pressure for the soil treated with foamed asphalt and sulfur foam asphalt. Also, it can be seen that the higher temperature lowers the modulus of resilience for all mixes due the fact that the mix tends to lose its strength with high temperature. Marl treated with foamed asphalt exhibits a higher resilient modulus value followed by sabkha and dune sand due to the strength and cohesion of particle. Similarly, in SFA mixes, marl soil exhibited higher resilient modulus followed by sabkha and dune sand soil. However, there is no dramatic change in resilient modulus value for marl soil treated with SFA compared with that treated with FA as shown in Figure 4.14. Also, for sabkha and dune sand there is no significant change in resilient modulus value compared to FA mixes.

Generally, resilient modulus testing indicated that SFA mixes has behavior comparable to FA mixes. Marl treated mixes showed the best behavior followed by sabkha and dune sand treated mixes. Temperature has reduced resilient modulus of all mixes significantly.

Table 4.10 shows the general regression equations for M_R with deviator stress, confining pressure and temperature. The general regression model for M_R value performed good correlation (R^2= 0.94) and is illustrated as follows:

$$M_R = 89.0 - 19.6\,A - 7.24\,B - 0.376\,T + 0.101\,\sigma_c + 4.80\,\sigma_d \quad (R^2 = 0.94) \tag{4.2}$$

Where,

M_R = resilient modulus

A = treatment type (FA or SFA).

B = material type (marl soil, sabkha and dune sand).

T = temperature (22 °C and 40 °C).

σ_c = confining pressure.

σ_d = deviator stress.

Table 4.10: General regression analysis for M_R versus T, σ_c and σ_d

Material	Treatment type	M_R	R^2
Marl	FA	M_R = 82.69 - 0.68 T - 0.22 σ_c+ 4.98 σ_d	0.96
	SFA	M_R = 59.98 - 0.68 T - 0.22 σ_c + 4.98 σ_d	
Sabkha	FA	M_R = 44.37 - 0.56 T - 0.089 σ_c + 4.73 σ_d	0.97
	SFA	M_R = 29.6251- 0.56 T - 0.089 σ_c + 4.73 σ_d	
Sand	FA	M_R = 22.38 - 0.34 T - 0.14 σ_c + 4.57 σ_d	0.97
	SFA	M_R = 10.60 - 0.34 T - 0.14 σ_c + 4.57 σ_d	

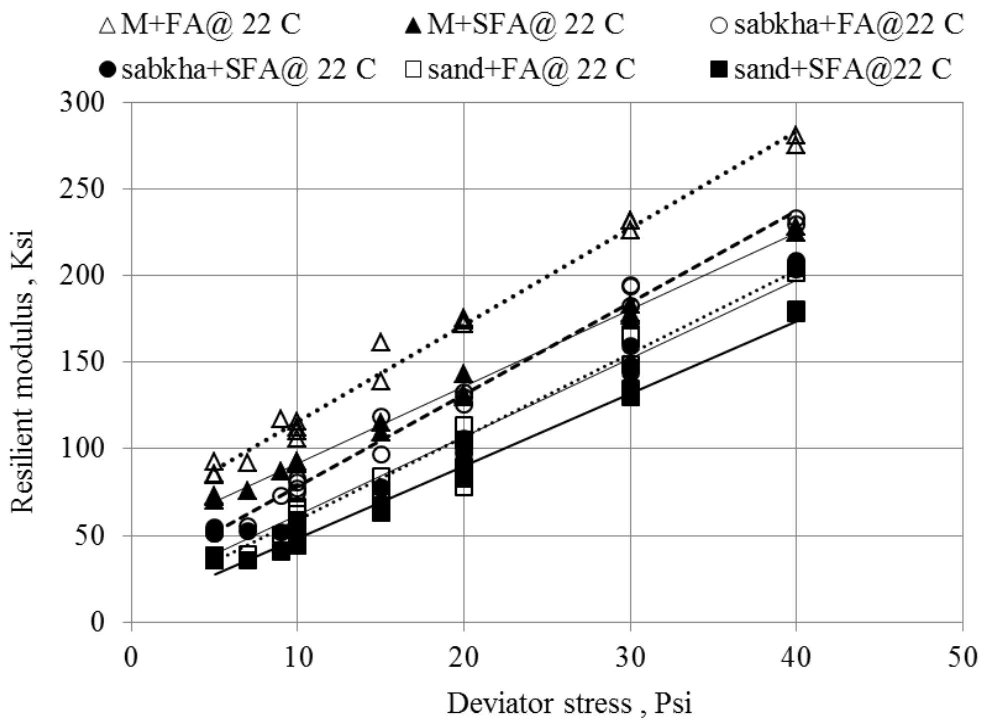


Figure 4.14: Variation of M_R with deviator stress for all mixes at 22 °C.

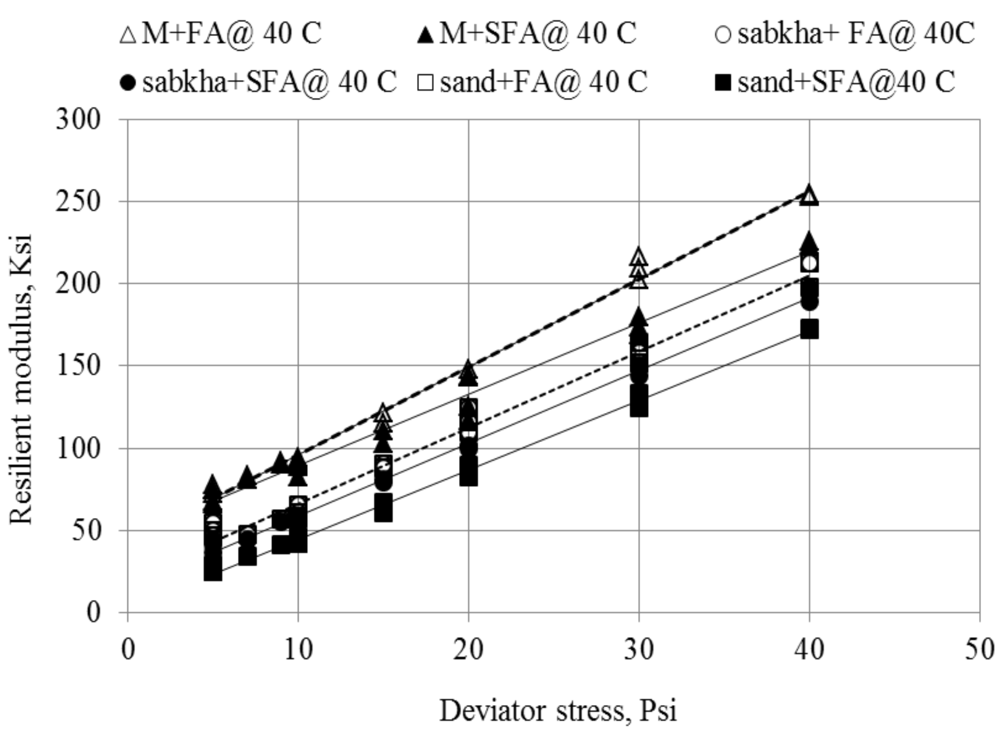


Figure 4.15: Variation of M_R with deviator stress for all mixes at 40 °C.

4.7.5 Dynamic Triaxial Test Results

Treated soils were subjected to dynamic triaxial test with three deviator stresses to simulate the field and to give a good prediction of rutting at a constant confining pressure of 10 psi and two temperatures, 22°C and 40ºC. The test results for FA and SFA mixes are presented in Figures 4.16-4.25.

The results indicated that the permanent deformation increased with increased level of stress and temperature for both FA and SFA mixes. Also, it can be seen that SFA mixes have more resistance to rutting because, it improves the properties of the material and enhanced the bonding between the particles.

Treated marl soil was subjected to dynamic triaxial test with three different levels of stress (80 psi, 70 psi and 60 psi) because of the ability of marl to withstand a high stress with a reasonable rutting life. A total of 54 samples of marl mixes was subjected to dynamic triaxial and the results of the permanent deformation for marl soil treated with FA and SFA at 22ºC and 40ºC are presented in Figures 4.16-4.17 respectively. The results indicated that the treated marl has good performance and rutting resistance due to the high cohesion of marl particle and high shear strength. Temperature variation did not significantly affect marl rutting as it did with sabkha and dune sand.

The treated sabkha soils were subjected to different deviator stresses (60 psi , 50 psi and 40 psi). These levels of deviator stress are less than that used for marl soil (80 psi) due to the fact that sabkha has a higher rutting tendency due to low shear strength.

The results of permanent deformation for sabkha treated with FA and SFA at 22°C and 40ºC are presented in Figures 4.18-4.21, respectively.

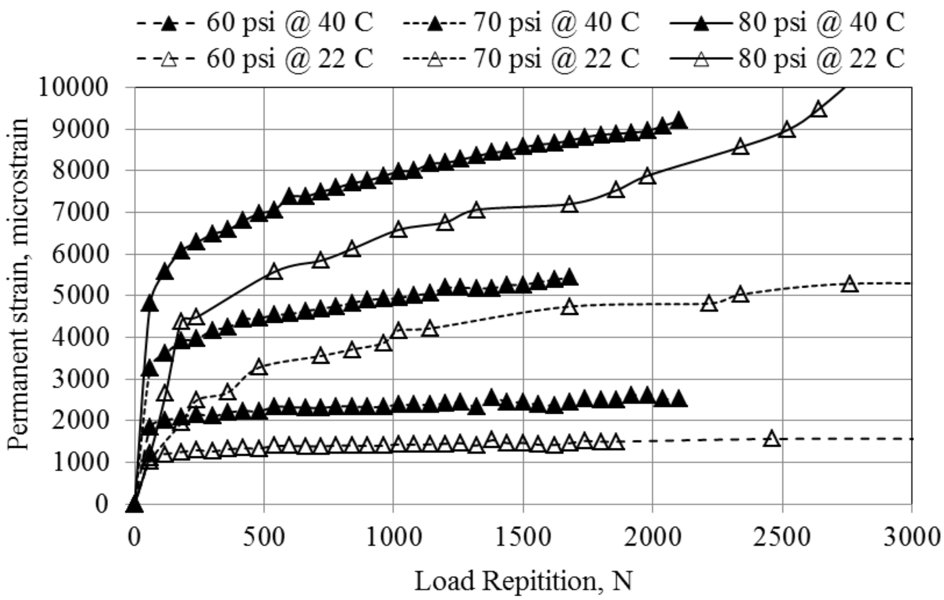


Figure 4.16: Rutting curves for marl soil treated with FA at different deviator stress .

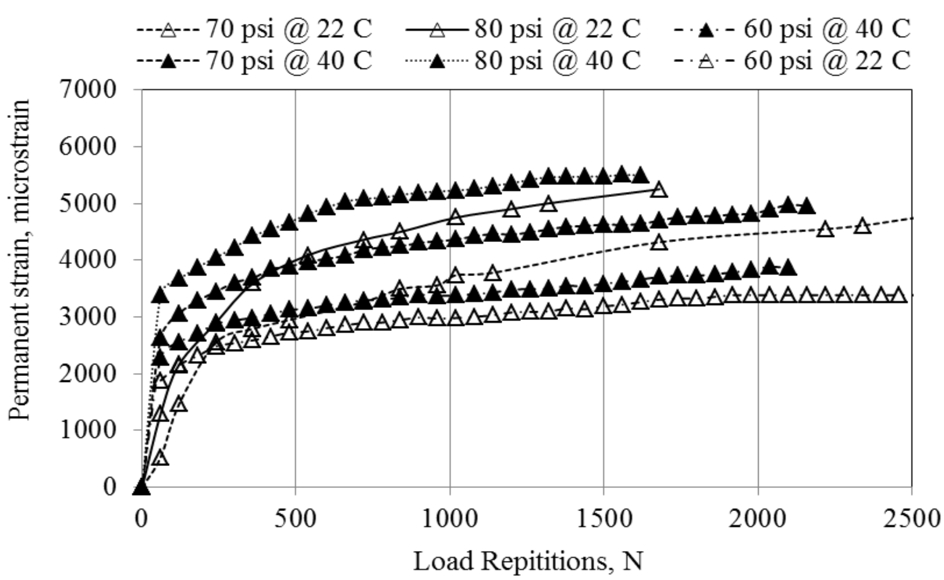


Figure 4.17: Rutting curves for marl soil treated with SFA at different deviator stress .

Sabkha soil treated with FA subjected to 40 psi deviator stress showed permanent strain of (5000 micro-strain) in the first stage at about 2000 cycles and changed to the steady stage. Similarly, at high deviator stress (50 psi and 60 psi) the permanent strain increased to about (1000 micro-strain and 15,000 micro-strain) at different number of cycles and continued increasing to reach the tertiary stage with more than 17,500 micro-strain deformation. On the other hand, the permanent strain for sabkha soil treated with FA at 40°C shows higher permanent strain with low number of cycles (500 cycles) as shown in Figure 4.19. This means that there is a dramatic increase of permanent strain with high temperature. Furthermore, the sabkha treated with SFA showed lower permanent strain with different level of stress as shown in Figures 4.20 and 4.21. However, there is a dramatic increase in permanent strain due to increase temperature as shown in Figure 4.21. It can be seen that, the permanent strain reached the steady stage at low number of cycles (400 cycles) which is very high compared to that at 22°C . From these results, it can be concluded that sabkha soil treated with SFA exhibited better rutting resistance and performance than that treated with FA. This is because the SFA enhanced the sabkha properties and increased the cohesion of sabkha particle.

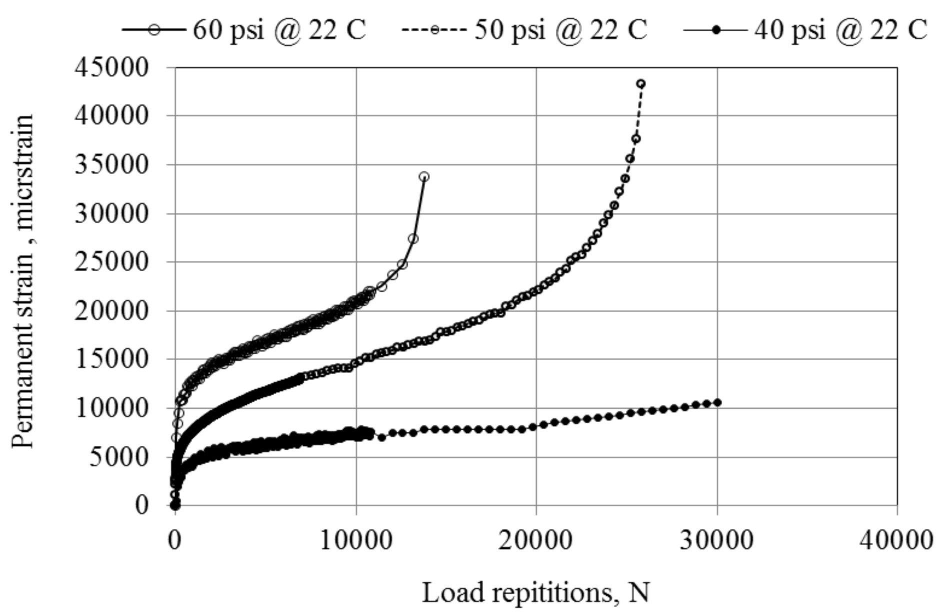


Figure 4.18: Rutting curves for sabkha treated with FA at different deviator stress.

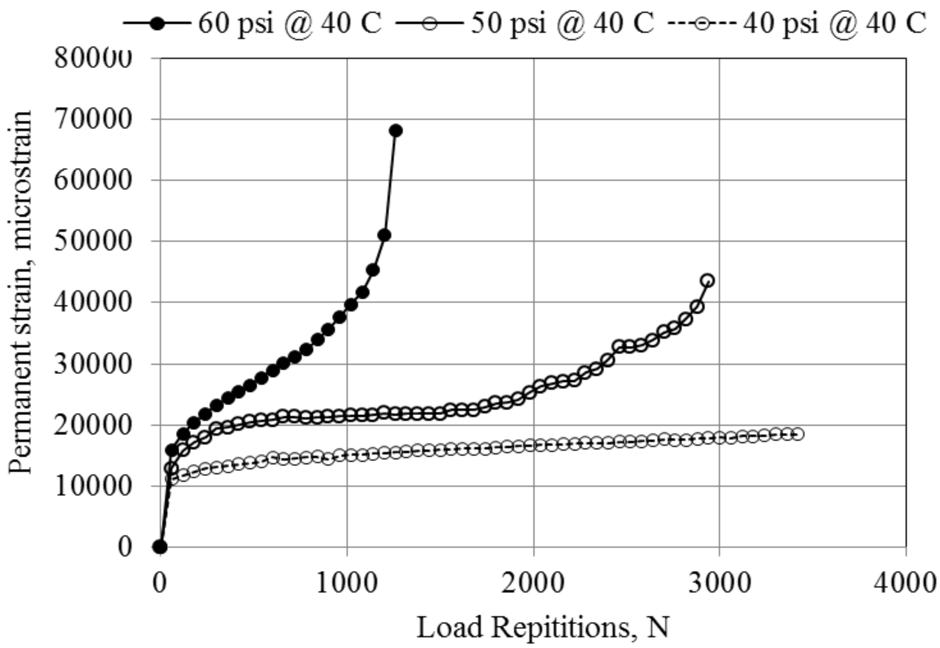


Figure 4.19: Rutting curves for sabkha treated with FA at different deviator stress .

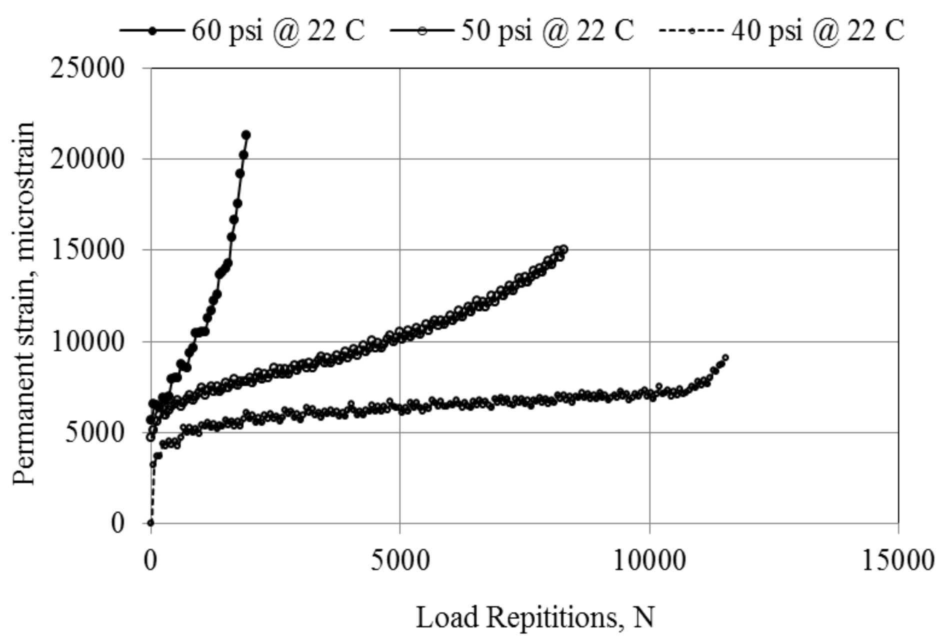


Figure 4.20: Rutting curves for sabkha treated with SFA at different deviator stress.

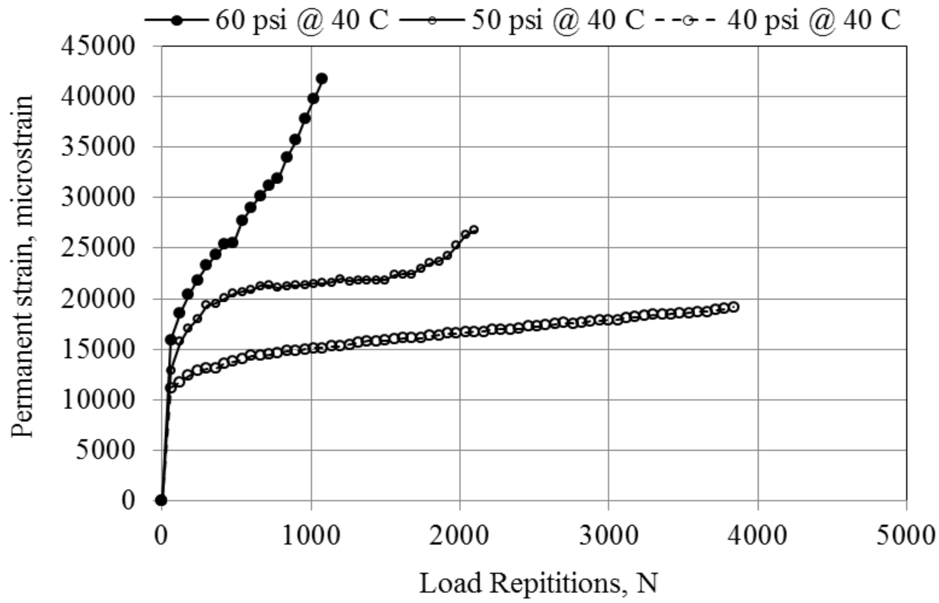


Figure 4.21: Rutting curves for sabkha treated with SFA at different deviator stress .

The dune sand treated with SFA and FA were tested with different stress levels (60 psi, 50 psi and 40 psi) with a confining pressure of 10 psi due to the fact that dune sand has higher rutting tendency due to low shear strength. Figure 4.22 shows the permanent strain for sand treated with FA, it can be seen that the dune sand treated with FA exhibited 5000 micro-strain deformation for the first stage with 40 psi @ 22°C at a low number of load repetitions (200 cycles) and continued to the steady stage with increased number of cycles. By increasing the deviator stress, the permanent strain increased and changed to steady stage up to failure with low number of load repetition. Furthermore, dune sand treated with FA at 40 °C has high change in permanent strain due to the fact that dune sand is more sensitive to temperature. It can be seen from Figure 4.23 that the permanent strain was very high and the treated sand reached the tertiary stage with a low number of load repetition (50 cycles) for all levels of stress. It can be concluded that the dune sand treated with FA can't be applicable to withstand traffic loads at a temperature of 40ºC.

Figures 4.24 and 4.25 show the permanent strain for dune sand treated with SFA at different temperature 22 ºC and 40ºC, respectively. It can be seen from these Figures that the treated sand reached the steady stage with low permanent strain for different levels of stress (40 psi, 50 psi and 60 psi) respectively with about 6500 cycles and reached the failure stage at low number of cycles.

On the other hand, the permanent strain dramatically increased with increasing temperature as shown in Figure 4.25. It can be seen that the treated sand changed from ductile to brittle material at 60 psi with a low number of repetitions. From these results it can be concluded that the dune sand treated with SFA exhibited lower rutting compared to sand treated with FA due to shear strength. However, treated dune sand is not suitable to resist traffic loads at high temperatures.

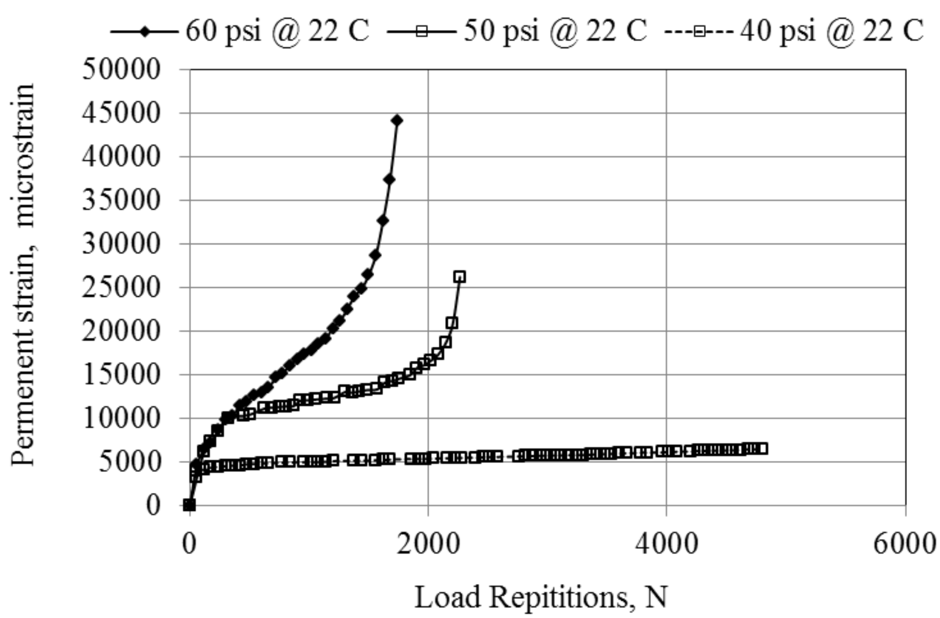


Figure 4.22: Rutting curves for sand soil treated with FA at different deviator stress .

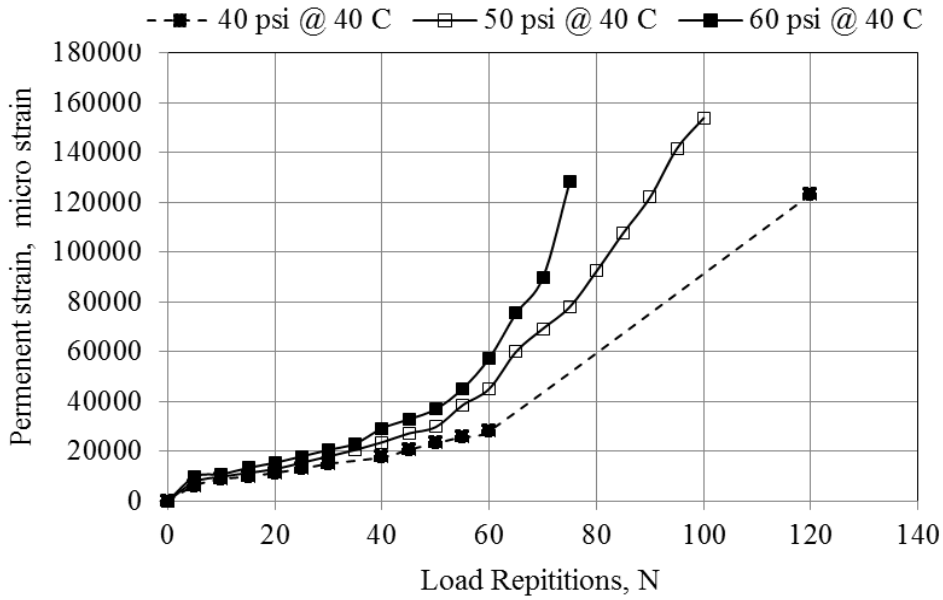


Figure 4.23: Rutting curves for dune sand treated with FA at different deviator stress.

Finally, we can conclude that SFA mixtures exhibited greater rutting resistance than FA mixes, this is due to the fact that SFA mixes have higher shear strength. Furthermore, treated marl soil has lower rutting followed by treated sabkha soil and dune sand due to the shear strength of each material.

The material properties α and μ were analyzed to create a model to calculate the general material properties at different levels of stress and temperature as shown in Table 4.11. This model was calculated and reported as follows:

$$\mu = -0.49 + 0.671\,A - 1.14\,B + 0.133\,T - 0.0148\,\sigma_d \qquad (R^2 = 0.81) \qquad (4.3)$$

$$\alpha = 0.98 - 0.045\,A + 0.038\,B - 0.0034T - 0.0017\,\sigma_d \qquad (R^2 = 0.74) \qquad (4.4)$$

Where,

μ and α = material properties.

A = treatment type (FA or SFA).

B = material type (marl soil, sabkha and dune sand).

T = temperature (22 °C and 40 °C).

σ_d = deviator stress.

Table 4.12 shows the general regression model of material properties μ and α versus temperature, deviator stress and treatment type for the investigated soil.

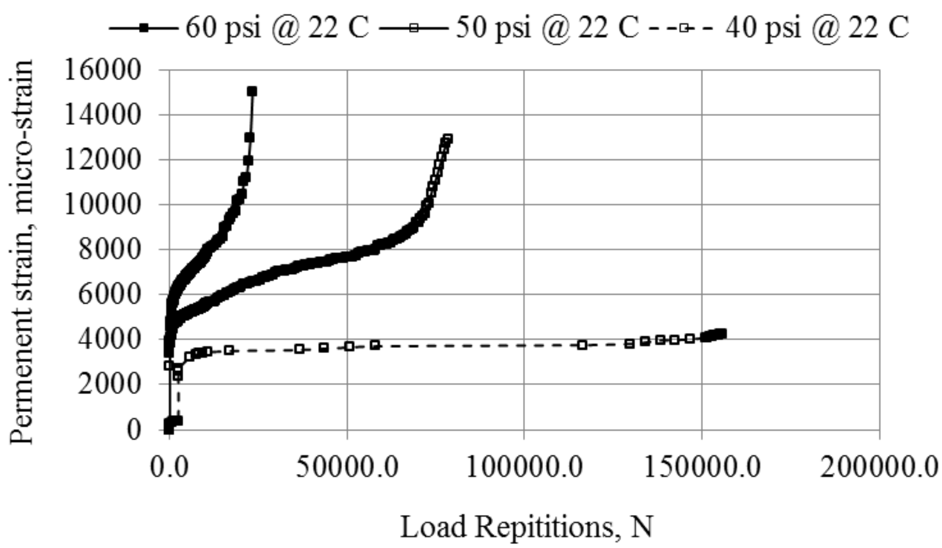


Figure 4.24: Rutting curves for dune sand treated with SFA at different deviator.

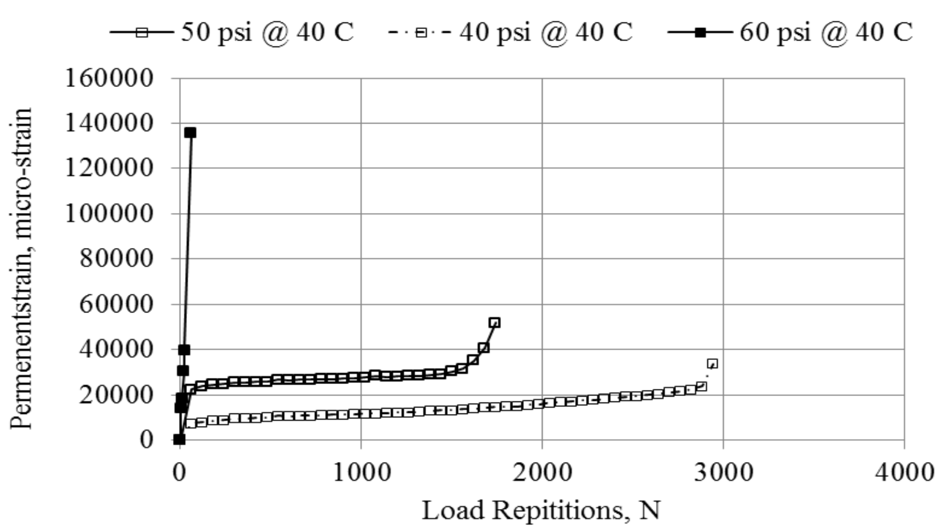


Figure 4.25: Rutting curves for dune sand treated with SFA at different deviator.

Table 4.11: Test variables for RLPD test for investigated mixes.

Type of mix	Temperature °C	σ_c, psi	σ_d, psi	a	b	R^2	μ	α
Marl + FA	40	10	80	2434	0.171	0.995	1.7569	0.8290
		10	70	1776	0.148	0.994	1.1756	0.8520
		10	60	1485	0.068	0.941	0.4828	0.9320
	22	10	80	998.4	0.263	0.956	1.1959	0.7370
		10	70	957.7	0.176	0.979	0.8192	0.8240
		10	60	666	0.123	0.979	0.4292	0.8770
Marl + SFA	40	10	80	1385	0.196	0.993	0.9122	0.8040
		10	70	1440	0.16	0.996	0.8295	0.8400
		10	60	1348	0.135	0.982	0.7094	0.8650
	22	10	80	831.7	0.142	0.996	0.4219	0.8580
		10	70	1160	0.132	0.983	0.5839	0.8680
		10	60	549.7	0.124	0.972	0.2803	0.8760
Dune sand + FA	40	10	60	2614	0.604	0.987	9.1065	0.3960
		10	50	2315	0.597	0.976	7.8188	0.4030
		10	40	2527	0.515	0.935	7.1904	0.4850
	22	10	60	544.4	0.503	0.935	1.2491	0.4970
		10	50	2280	0.242	0.976	2.6112	0.7580
		10	40	2196	0.123	0.987	1.3372	0.8770
Dune sand + SFA	40	10	60	2903	0.801	0.983	8.9080	0.1990
		10	50	4336	0.267	0.650	4.5398	0.7330
		10	40	2712	0.215	0.948	2.3527	0.7850
	22	10	60	544.4	0.514	0.992	0.9376	0.4860
		10	50	2280	0.242	0.954	1.9495	0.7580
		10	40	2196	0.123	0.935	1.0184	0.8770
Sabkha + FA	40	10	60	4167	0.309	0.955	4.8749	0.6910
		10	50	5372	0.219	0.984	4.7807	0.7810
		10	40	5316	0.152	0.960	3.5805	0.8480
	22	10	60	2442	0.26	0.971	3.1541	0.7400
		10	50	1326	0.25	0.996	1.7385	0.7500
		10	40	924.8	0.168	0.860	0.8706	0.8320
		10	20	1913	0.144	0.960	1.8965	0.8560
Sabkha + SFA	40	10	60	3112	0.216	0.955	2.4885	0.7840
		10	50	3168	0.177	0.870	2.1982	0.8230
		10	40	2968	0.148	0.922	1.8470	0.8520
	22	10	60	1151	0.304	0.929	1.1921	0.6960
		10	50	2261	0.171	0.935	1.4185	0.8290
		10	40	2061	0.132	0.902	1.0928	0.8680

Table 4.12: Regression analysis of material properties versus T, σ_d, treatment type for investigated soil.

Material	Treatment type	μ	R^2	α	R^2
Marl	FA	$\mu = -1.72 + 0.02\ T + 0.03\ \sigma_d$	0.76	$\alpha = 1.10 + 0.00076\ T - 0.004\ \sigma_d$	0.56
	SFA	$\mu = -2.07 + 0.02\ T + 0.03\ \sigma_d$		$\alpha = 1.11 + 0.00076\ T - 0.004\ \sigma_d$	
Sabkha	FA	$\mu = -2.49 + 0.09\ T + 0.05\sigma_d$	0.84	$\alpha = 1.06 + 0.00059\ T - 0.006\ \sigma_d$	0.84
	SFA	$\mu = -3.95 + 0.09\ T + 0.05\sigma_d$		$\alpha = 1.09 + 0.00059\ T - 0.006\ \sigma_d$	
Sand	FA	$\mu = -9.12 + 0.28\ T + 0.10\sigma_d$	0.82	$\alpha = 1.83 - 0.011\ T - 0.018\ \sigma_d$	0.76
	SFA	$\mu = -10.74 + 0.28\ T + 0.10\sigma_d$		$\alpha = 1.90 - 0.011\ T - 0.018\ \sigma_d$	

4.7.6 Test on Subgrade Materials

The subgrade materials consist of compacted soils (sabkha, dune sand and marl soils) which have the same characteristics that are mentioned in Chapter 3. The following tests were carried out on these collected sub grade materials.

4.7.7 Dynamic Triaxial Test Results

The untreated local soils were subjected to dynamic triaxial tests to determine the material properties. The results indicated that the compacted marl soil exhibits more resistance for rutting than other material. The material properties for subgrade were calculated and reported in Table 4.13.

Table 4.13: Summary of triaxial test results on sub grade material

Subgrade material	Test Temperature °C	Confining stress, σ_c (psi)	Deviator stress, σ_d (psi)	μ	α	R^2
Marl soil	25	5	10	0.07	0.90	0.991
Sabkha soil	25	5	10	0.12	0.84	0.996
Dune sand	25	5	10	1.20	0.89	0.992

4.8 Data Analysis

The effects of modified sulfur foam asphalt and foamed asphalt as stabilizer technology for local soil mixes were analyzed statistically using the data obtained from the different tests performed on modified mixes. A statistical methodology in the form of analysis of variance using two-factor factorial analysis is performed to verify the significance and reliability of the main variables (FA and SFA), as well as the material type (sabkha, dune sand and marl soil) on mixture test results. The two factor analyses used in this research are treatment type (FA and SFA) and material types (sabkha, dune sand and marl soil).

The null hypothesis of an ANOVA is that the population means of all the treatments are equal (the treatments have equal effects). The alternative hypothesis is that at least one population means is significantly different from the others. The typical Type I error rate of 0.05 was used throughout the analysis. Thus, when the level of significance, or p-value, was less than or equal to 0.05, the null hypothesis was rejected, and the alternative hypothesis was accepted. When the p-value was greater than 0.05, insufficient evidence existed to accept the null hypothesis. A summary of the findings is given in the following section. The abstract of the results is presented in detail, but the complete Minitab print out is presented in Appendix A.

4.9 Analysis of Variance (Anova.)

The results of indirect tensile strength for local soil stabilized with FA and SFA are presented in Table 4.14. The dry and wet results represent the unconditioned and conditioned specimens respectively. Testing was conducted under the same procedures.

Table 4.14: Indirect tensile strength test results.

	Treatment Type			
	FA		SFA	
Material Type	Dry ITS	Soaked ITS	Dry ITS	Soaked ITS
Marl	528.24	360.18	665.3	544.8
	496.79	370.15	640.0	491.2
	569.31	376.13	635.9	539.9
Sabkha	448.0	183.43	320	181.43
	425.8	178.02	310	178.02
	410.0	198.10	305	144.10
Dune Sand	81.0	56.856	88.0	58.000
	85.0	56.856	91.0	62.000
	78.0	56.856	76.0	52.000

The analysis results of ITS test for treated soils are shown in Tables 4.15-4.16. The results indicated that the treatment type (FA and SFA) significantly affects the dry and soaked ITS of treated marl soil and sabkha soil. This means that the sulfur foam asphalt has a significant improvement on the treated soil ITS when compared with foamed asphalt.

Table 4.15: Results of dry ITS ANOVA at 5% significance level.

Material type	Factors/Additives	Calculated F_{value}	P-value	Comment
Marl	Type of additives FA or SFA	25.44	0.007	Significant
Sabkha	Type of additives FA or SFA	95.93	0.001	Significant
Dune Sand	Type of additives FA or SFA	0.54	0.505	Insignificant

Table 4.16 shows the analysis results of soaked ITS for treated soils. From the data in Table 4.16 there is a significant improvement in the ITS value for marl soil (360.18 kpa and 544.14 kpa) and insignificant improvement in the ITS value for sabkha and dune sand due to particle bonding as shown in Table 4.14.

Table 4.16: Results of soaked ITS ANOVA at 5% significance level.

Material type	Factors/Additives	Calculated F_{value}	P-value	Comment
Marl	Type of additives FA or SFA	82.18	0.001	Significant
Sabkha	Type of additives FA or SFA	3.23	0.147	Insignificant
Dune Sand	Type of additives FA or SFA	0.03	0.877	Insignificant

Statistical analysis of Marshall Stability test results for both dry and soaked stability are shown in Tables 4.17-4.18. The results indicated that the treatment type has a significant effect on marl and dune sand soil. It can be concluded that the treatment (asphalt type) significantly affects the stability of the treated soil (marl and sand, Pv < 5%). This means that the sulfur foam asphalt has a significant improvement on the marl and sand stability. On the other hand, the analysis result shows that there is no significant effect on dry stability value for treated sabkha soil due to cementing particles.

The analysis of resilient modulus (M_R) data for marl treated with FA and SFA were evaluated. The analysis result shows that treatment type, temperature, confining pressure and deviator stress have significant effects on M_R value in stabilized mixes, whereas the deviator stress has a more pronounced effect in treated samples. The analysis results for resilient modulus (M_R) are reported in Table 4.19.

The material properties α and μ were analyzed using linear model analysis to determine the effects of stress and temperature. The analysis results of material properties are presented in Tables 4.20-4.21.

Table 4.17: Results of dry stability ANOVA at 5% significance level.

Material type	Factors/Additives	Calculated F_{value}	P-value	Comment
Marl	Type of additives FA or SFA	12.82	0.023	Significant
Sabkha	Type of additives FA or SFA	0.18	0.691	Insignificant
Dune Sand	Type of additives FA or SFA	101.94	0.001	significant

Table 4.18: Results of soaked stability ANOVA at 5% significance level.

Material type	Factors/Additives	Calculated F_{value}	P-value	Comment
Marl	Type of additives FA or SFA	1.75	0.257	significant
Sabkha	Type of additives FA or SFA	26.21	0.007	significant
Dune Sand	Type of additives FA or SFA	12.23	0.025	significant

Table 4.19: Results of M_R value ANOVA at 5% significance level.

Material type	Factors/Additives	Calculated F_{value}	P-value	Comment
Marl	Type of additives FA or SFA	11.78	0.001	Significant
	Temperature	40.61	0.000	Significant
	Confining Pressure	32.76	0.000	Significant
	Deviator stress	145.64	0.000	Significant
Sabkha	Type of additives FA or SFA	10.27	0.002	Significant
	Temperature	21.46	0.000	Significant
	Confining Pressure	35.23	0.000	Significant
	Deviator stress	159.05	0.000	Significant
Dune Sand	Type of additives FA or SFA	4.03	0.049	Significant
	Temperature	15.01	0.000	Significant
	Confining Pressure	43.61	0.000	Significant
	Deviator stress	165.10	0.000	Significant

Table 4.20: Results of µ value ANOVA at 5% significance level.

Material type	Factors/Additives	Calculated F_{value}	P-value	Comment
Marl	Treatment	5.84	0.046	Significant
	Temperature	5.91	0.045	Significant
	Deviator stress	5.66	0.035	Significant
Sabkha	Treatment FA or SFA	13.3	0.008	Significant
	Temperature	18.38	0.004	Significant
	Deviator stress	2.48	0.153	insignificant
Dune Sand	Type of additives FA or SFA	2.52	0.156	insignificant
	Temperature	25.94	0.001	Significant
	Deviator stress	1.43	0.301	insignificant

Table 4.21: Results of α value ANOVA at 5% significance level.

Material type	Factors/Additives	Calculated F_{value}	P-value	Comment
Marl	Treatment	0.19	0.674	Insignificant
	Temperature	0.36	0.568	Insignificant
	Deviator stress	4.16	0.064	Insignificant
Sabkha	Treatment FA or SFA	3.98	0.086	Insignificant
	Temperature	0.37	0.563	Insignificant
	Deviator stress	16.23	0.002	significant
Dune Sand	Type of additives FA or SFA	0.98	0.355	insignificant
	Temperature	8.65	0.022	Significant
	Deviator stress	9.33	0.011	significant

4.10 Summary

A comprehensive testing was conducted on local soils treated mixes. Firstly, different percentages of foamed were mixed with investigated local soils to determine the optimum binder content by ITS test. Marshall stability, resilient modulus and permanent deformation tests were performed at the optimum binder content. Resilient modulus and permanent deformations tests were conducted at 25°C and 40 °C with different stress levels. The tests were used with a loading duration of 0.1 sec and load frequency of 60 cycles per minute. Analysis of variance (Anova) was performed to estimate the effect of treatment type on material types and it indicated that there are significant effects.

CHAPTER 5

PERMANENT DEFORMATION MODELING

5.1 Modeling

5.1.1 Introduction

This chapter focus on the permanent deformation modeling. In this research six stabilized mixtures (three replicate for each one) were prepared to evaluate the permanent deformation rutting behavior at 25°C and 40°C temperature and three different stress levels under repeated load test.

5.1.2 Model Selection

Many researchers have attempted to develop a comprehensive and realistic model for permanent deformation. Their efforts resulted in a realistic, simple sound solution. These models are available in several forms as tabular or graphical data, or as computer programs such as vesys w5 program.

Since computer programs have a greater capability and versatility, vesys program was selected for use in this research.

5.1.3 Material characterizations

The foamed asphalt mixes properties were obtained from laboratory results as discussed in Chapter 4 (Table 4.11). The material properties, Alpha (α) & Mu (μ), have been computed in this section from the intercept and slope of the permanent deformation trend line.

3D move analysis program were used to calculate stresses and strains in the pavement using non-linear relationships between stress and resilient modulus for each material. From the pavement stress analysis, the mean principal stress (p) and deviatoric stress (q) under the center of the load were calculated for input into a vesys 5w for the calculation of rut depth.

To determine material properties the following steps were followed.

1- The material properties Alpha (α) & Mu (μ) were calculated from permanent deformation behavior for each material with10 psi confining pressure and different levels of deviator stress at 22°C and 40°C temperatures (3 samples for each one).
2- The material properties α & μ for each material were plotted against deviator stress and reported in appendix.
3- Regression equation to determine the material properties was calculated from the deviator stress and temperature relation.
4- The resilient modulus is calculated according to AASHTO T-307 for each material (15 values for each material) as described in chapter 3 (Table 3.2).
5- M_R was calculated by linear regression from the relationship between M_R and deviator stress at 10 psi confining pressure.
6- Pavement analysis was done using 3D move analysis to determine the pavement stress.

7- The calculated pavement stress was used to determine the material properties α & μ from linear regression between the α & μ with deviator stress.

Material properties calculated depending on pavement response which was used to predict the rutting model and developing design chart. Table 5.1 shows regression equation for material properties and pavement response stress of each stabilized material at 22°C and 40°C temperature.

Table 5.1: Regression equation for material properties.

Type of mix	Temperature °C	μ	α
Marl+FA	40	$\mu = 0.0637\ \sigma_d - 3.321$	$\alpha = 1.2315 - 0.0052\ \sigma_d$
	22	$\mu = 0.0383\ \sigma_d - 1.8686$	$\alpha = 1.30 - 0.0070\ \sigma_d$
Marl+SFA	40	$\mu = 0.0101\ \sigma_d + 0.1071$	$\alpha = 1.0498 - 0.003\ \sigma_d$
	22	$\mu = 0.022\ \sigma_d - 1.015$	$\alpha = 0.930 - 0.0009\ \sigma_d$
Dune sand + FA	40	$\mu = 0.0958\ \sigma_d + 3.2484$	$\alpha = 0.6505 - 0.0044\ \sigma_d$
	22	$\mu = 0.04579\ \sigma_d - 0.4285$	$\alpha = 1.660 - 0.0190\ \sigma_d$
Dune sand + SFA	40	$\mu = 0.3278\ \sigma_d - 11.121$	$\alpha = 2.037 - 0.0293\sigma d$
	22	$\mu = 0.04\ \sigma_d - 0.5899$	$\alpha = 1.684 - 0.019\ \sigma_d$
Sabkha + FA	40	$\mu = 0.0647\ \sigma_d + 1.1758$	$\alpha = 1.1658 - 0.0078\ \sigma_d$
	22	$\mu = 0.1142\ \sigma_d - 3.7877$	$\alpha = 1.004 - 0.0046\ \sigma_d$
Sabkha + SFA	40	$\mu = 0.0321\ \sigma_d + 0.5741$	$\alpha = 0.9897 - 0.0034\ \sigma_d$
	22	$\mu = 0.0341\ \sigma_d - 0.2775$	$\alpha = 1.2276 - 0.0086\ \sigma_d$

5.1.4 Design Criteria

Design procedures based on multilayer elastic theory limit stress caused by rutting at the surface of subgrade. The allowable permanent strain is considered a good indicator for rutting. The relationship between the permanent deformation as a function of the number of 18 kip (80 in) equivalent axle load repetitions was used in this research. Rutting curves were developed to predict rutting in the stabilized layers .

5.1.5 Cases Analyzed

In Saudi Arabia subgrade soil is usually one of three materials marl, sand and sabkha. Three cases were analyzed as follows.

1- First case (Marl subgrade)

This case consists of a system containing three layers having 2 inches of HMA for local streets and base coarse of marl improved with two stabilizers (FA and SFA) over untreated marl sub grade (CBR=25) as shown in Figure 5.1. Since marl is more resistance to rutting compared to other materials, there is no need for a sub base layer.

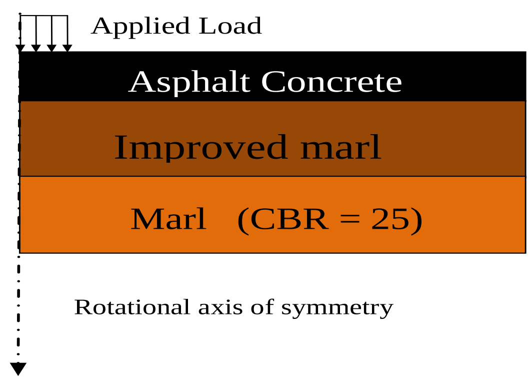
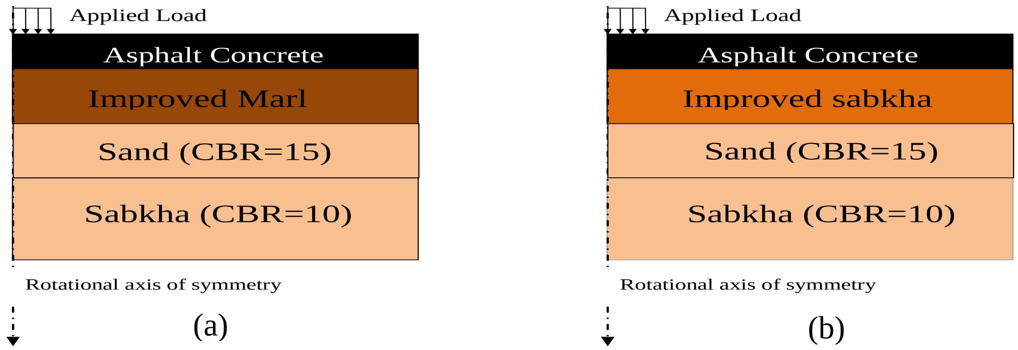


Figure 5.1: First case analyzed for marl (3-layer system).

2- Second case (Sabkha subgrade)

This case consists of two systems as shown in Figure 5.2. First system consists of four layer system having 2 inches of HMA and a base coarse of marl improved with two stabilizers (FA and SFA) over 6 inch sand sub-base to distribute the load and cut off water table as drainage layer as shown in Figure 5.2(a). The second system is a four layer system having 2 inches HMA and base coarse of sabkha improved with two stabilizers (FA and SFA) over a 6 inch sand (CBR=15) subbase as shown in Figure 5.2(b). Sabkha soil can’t use as a full depth pavement due to its water sensitivity and capillary property, especially in high water table sabkha area.

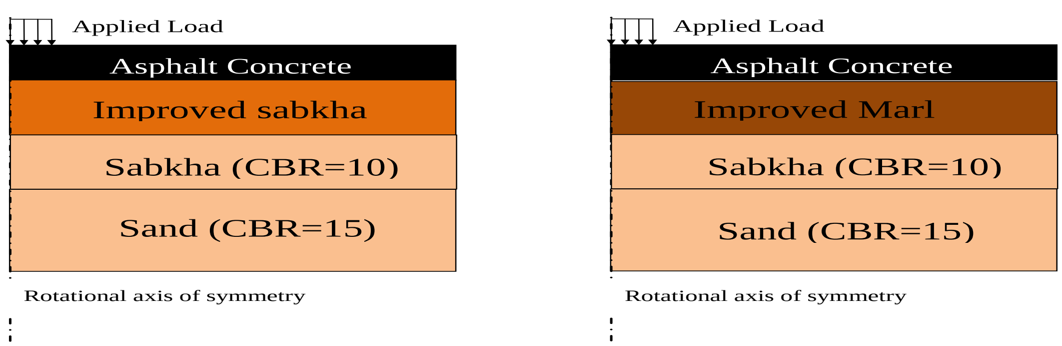


(a) (b)

Figure 5.2: Second case analyzed (4-layer system).

3. Third case (Sand subgrade)

For sand subgrade, two systems were analyzed. The first one is a system containing four layers having 2 inches of HMA for highways and a base coarse of sabkha improved with two stabilizers (FA and SFA) over 6 inches untreated sabkha (CBR=10) subbase as shown in Figure 5.3 (a). The second system is a four layers system having 2 inches of HMA for highways and a base coarse of improved marl with two stabilizers (FA and SFA) over 6 inches sabkha subbase (CBR=10) above the untreated sand (CBR=15) subgrade as shown in Figure 5.3 (b).



(a) (b)

Figure 5.3: Third case analyzed (4-layer system).

5.1.6 Design Charts

The results of the structural analysis are reported in the form of design charts as shown in Figures 5.4-5.7. The stabilized layer thickness is plotted vs. allowable total traffic of equivalent 18 kip axle load (EAL) for the local stabilized material.

Rutting was the controlling criteria (i.e. Pavement rut was limited to 1.0 inch (2. 54 cm)).

Three materials were evaluated, including marl treated with (FA and SFA) mix, sabkha treated with (FA and SFA) and dune sand treated with (FA and SFA) at two temperatures 22º C and 40º C.

5.1.7 Implementation

The design charts specified in Figures 5.4 to 5.7 were developed for materials at two different temperatures 22 °C and 40 °C. The results were compared with each stabilized for each case.

Comparison between marl improved with FA and that improved with SFA, indicated that marl treated with SFA has higher design value than treated FA as shown in Figure 5.4. Similarly, for second case, the base coarse constructed of marl treated with SFA has higher design value followed by marl treated with FA, sabkha treated with SFA and sabkha treated with FA as shown in Figure 5.5. In addition, for third case, it has the same. It can be seen that the base coarse stabilized with SFA exhibited higher design value than FA due to shear strength.

The use of sulfur foamed treated materials for road bases in Saudi Arabia provides an economical mean for road construction. These charts can be used for locally constructed roads , usually require a hot asphalt thickness of 2 in hot asphalt for 3 layer system highways over untreated marl (CBR=25), sabkha(CBR =10) and dune sand (CBR=15) subgrade and 2 inch hot asphalt for 4-layer system with 6 inch subbase according to the cases analyzed.

The abundant of local soils in Sadia Arabia will led to reduce the use of crushed aggregate and substantially will reduce the pavement costs. Since, these soils dominated more than 50% of Saudia Arabia area. Moreover, it will enable the use of locally available marl, sabkha and dune sand which are available at lower costs. In addition, the sulfur will reduce the asphalt up to 30% . Therefore, it is recommended to use sulfur foam asphalt, especially where there is a lack of good low cost aggregate for base stabilization and especially for construction of low volume and agricultural roads.

5.2 Summary

This chapter describes an effort to adapt test results to local field use. The material properties were obtained from laboratory experiments . limiting surface rutting to 1″ was used to develop design charts. Three local soils (marl, sand and sabkha) treated with foamed asphalt and sulfur foam asphalt mixes at two temperatures (22° C and 40 ºC) were considered for Vesys 5w program. Design charts can be used for local roads in Saudi Arabia.

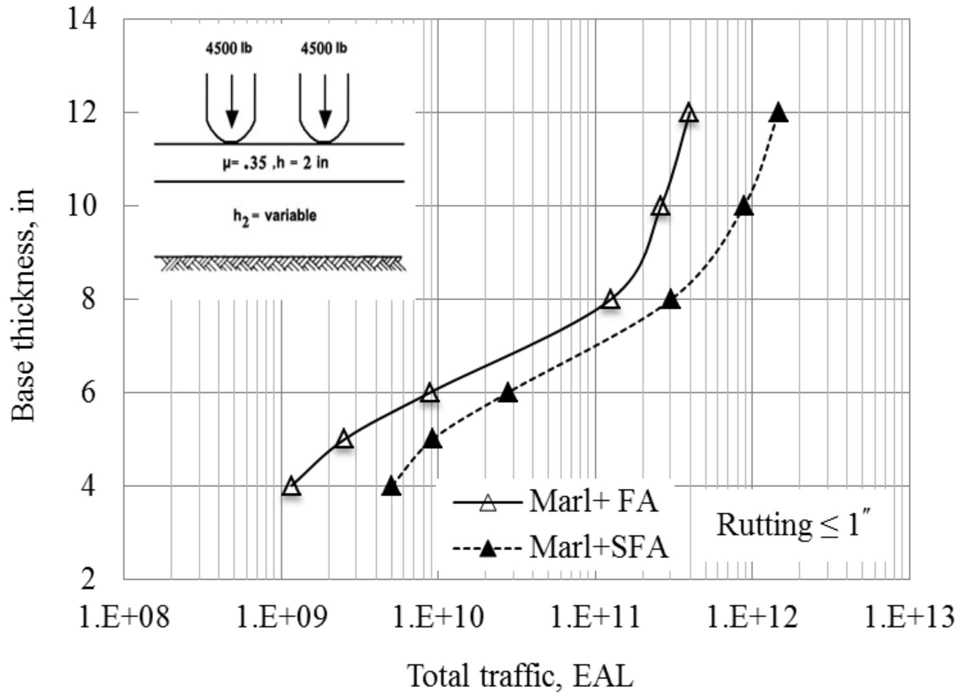


Figure 5.4: Relationship between pavement thickness and total traffic for first case.

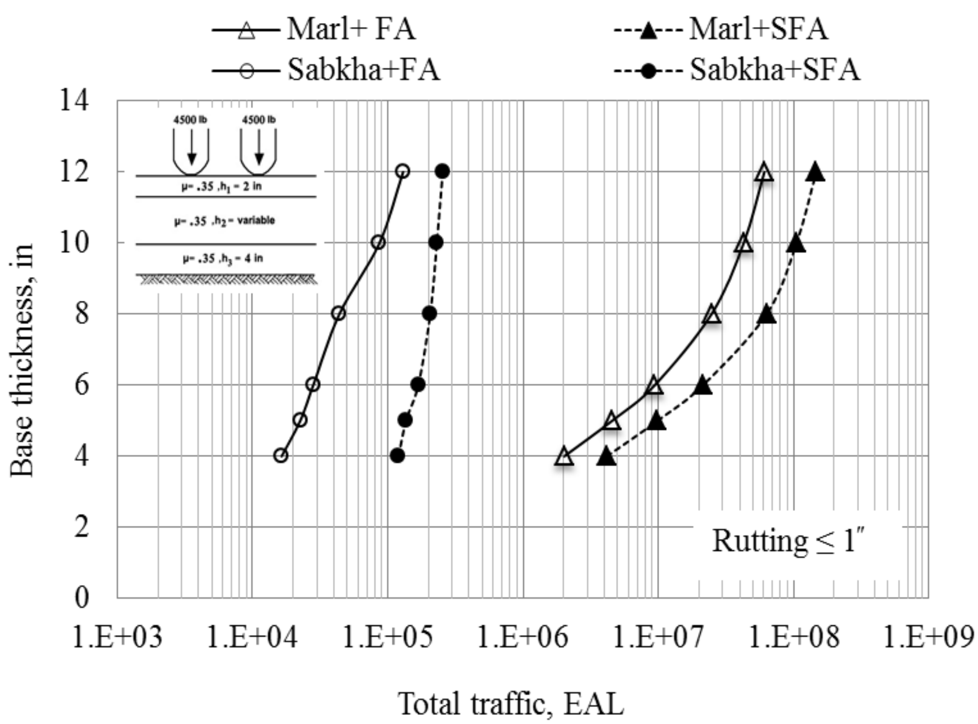


Figure 5.5: Relationship between pavement thickness and total traffic for second case.

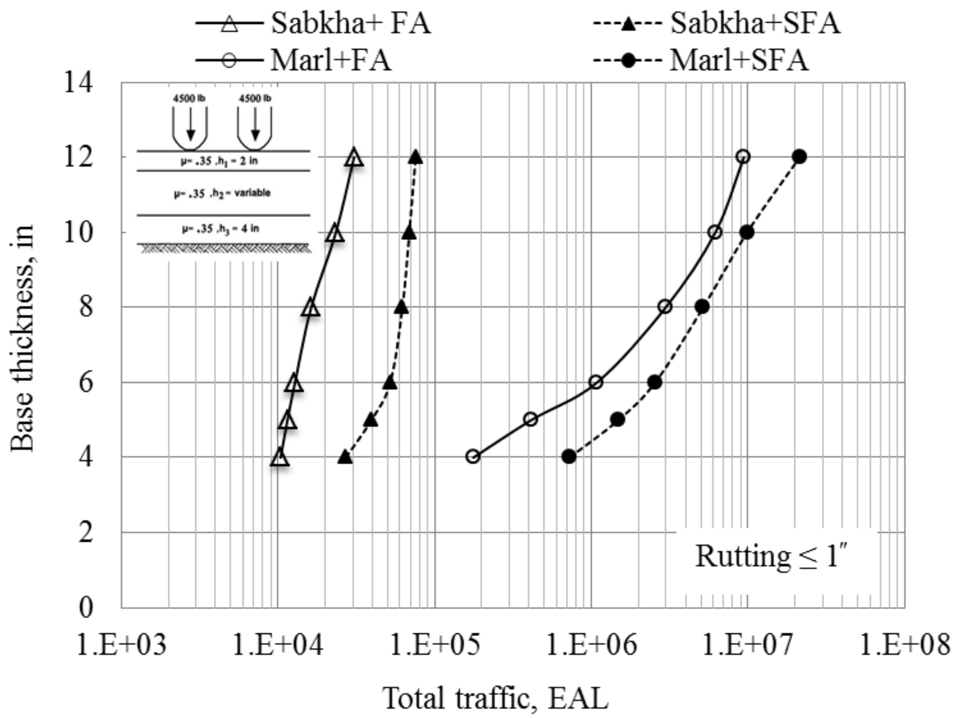


Figure 5.6: Relationship between pavement thickness and total traffic for third case.

CHAPTER 6

CONCLUSION AND RECOMMENDATIONS

6.1 Conclusion

This research presents a laboratory study to evaluate modified sulfur foam asphalt to stabilize local soils (marl, sabkha, and dune sand). The performance of the SFA mixtures were compared to FA mixtures with regard to ITS, stability, M_R and permanent deformation (or rutting).

Based on the experimental test results and the subsequent statistical analyses following conclusions and recommendations are made:

1- Sulfur foam asphalt was successfully produced using a laboratory scale asphalt foaming device called WLB10, produced by Wirtgen.

2- Sulfur foam asphalt (SFA) treatment significantly improved the overall performance of treated mixes and can be effectively used for the stabilization of local soils.

3- Foamed asphalt (FA) treatment gives satisfactory results. However, according to the results of ITS, stability, M_R and permanent deformation tests, FA mixes exhibited inferior performance as compared to SFA mixes.

4- The effect of treatment type, regardless to the material type on the performance of treated mixes was significant in most of all cases. However, the statistical analysis revealed that the treatment type has significant effect on ITS, stability and M_R value.

5- Both SFA and FA mixes satisfied the required stability recommended by Asphalt Institute.

6- The modified sulfur foam asphalt has significant effect on shear strength of investigated soil. It increased the angle of interaction and cohesion of investigated soil. As result, the investigated soil becomes more resistance to rutting and performance as compared to foamed mixes.

7- The modified sulfur foam asphalt mixtures and foamed asphalt mixture can be used successfully to improve the quality of local soil materials for road base construction.

8- The best performance was attained by marl treated with SFA followed by sabkha and dune sand.

6.2 Recommendations

Significant recommendation resulting from this research include:

1- Modified foamed sulfur asphalt should be used for stabilizing local soil marl, sabkha and dune sand for road base construction whenever there is a lack of good quality of aggregate.

2- Modified foamed sulfur asphalt and foamed asphalt mixes should be introduced in Saudi road specification as a construction material.

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APPENDIX

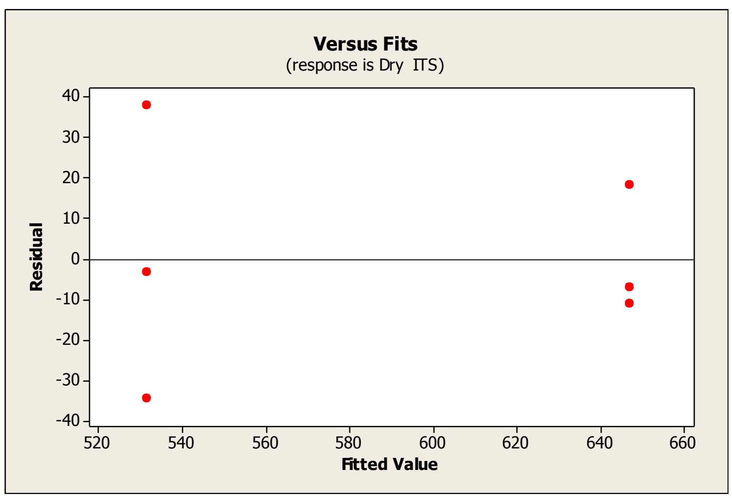
Minitab printout: ITS

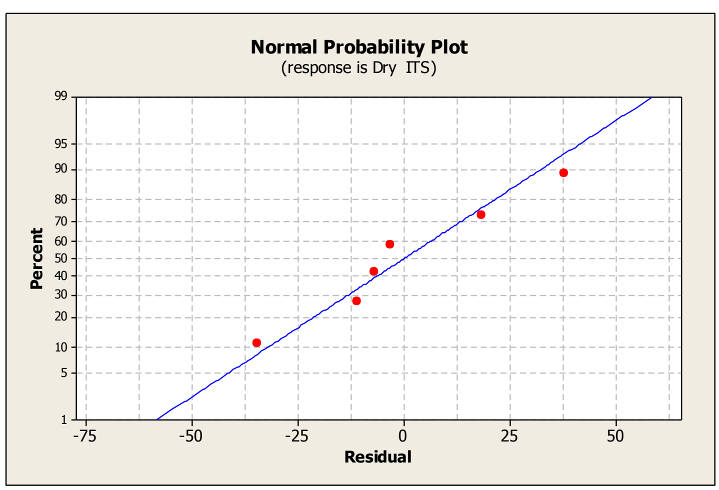
Minitab ANOVA printout: Dry ITS of marl soil

One-way ANOVA: Dry ITS versus Treatment

```
Source      DF     SS     MS      F      P
Treatment    1  20054  20054  25.44  0.007
Error        4   3153    788
Total        5  23207

S = 28.08   R-Sq = 86.41%   R-Sq(adj) = 83.02%
```



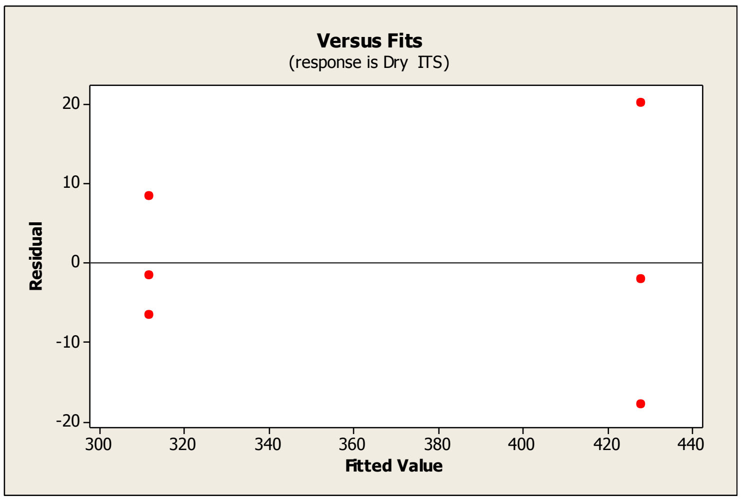


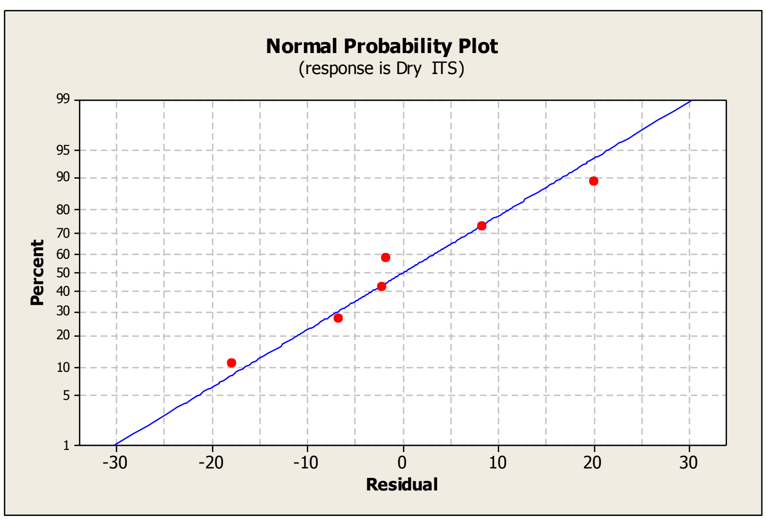
Minitab ANOVA printout: Dry ITS of sabkha soil

One-way ANOVA: Dry ITS versus Treatment

Source	DF	SS	MS	F	P
Treatment	1	20277	20277	95.93	0.001
Error	4	845	211		
Total	5	21122			

S = 14.54 R-Sq = 96.00% R-Sq(adj) = 95.00%.



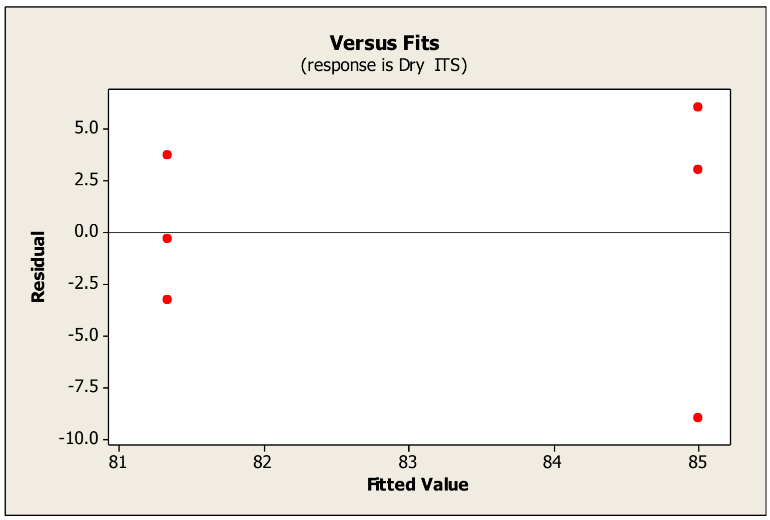


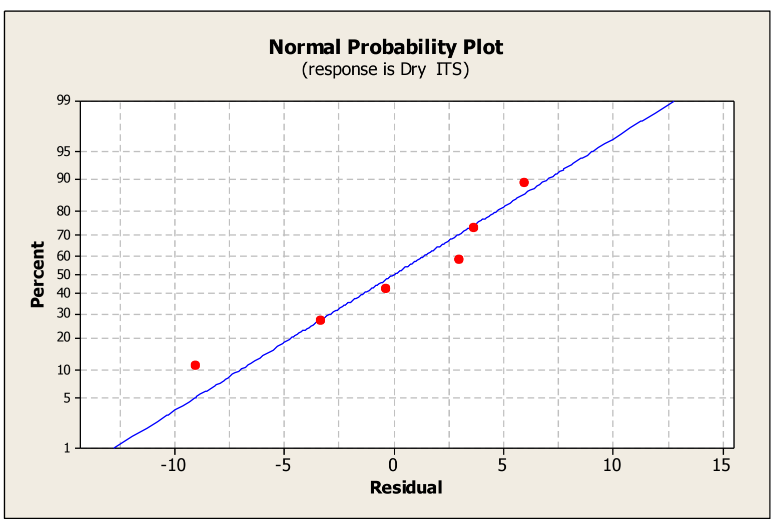
Minitab ANOVA printout: Dry ITS for sand soil

One-way ANOVA: Dry ITS versus Treatment.

```
Source     DF     SS    MS     F      P
Treatment   1   20.2  20.2  0.54  0.505
Error       4  150.7  37.7
Total       5  170.8

S = 6.137   R-Sq = 11.80%   R-Sq(adj) = 0.00%.
```



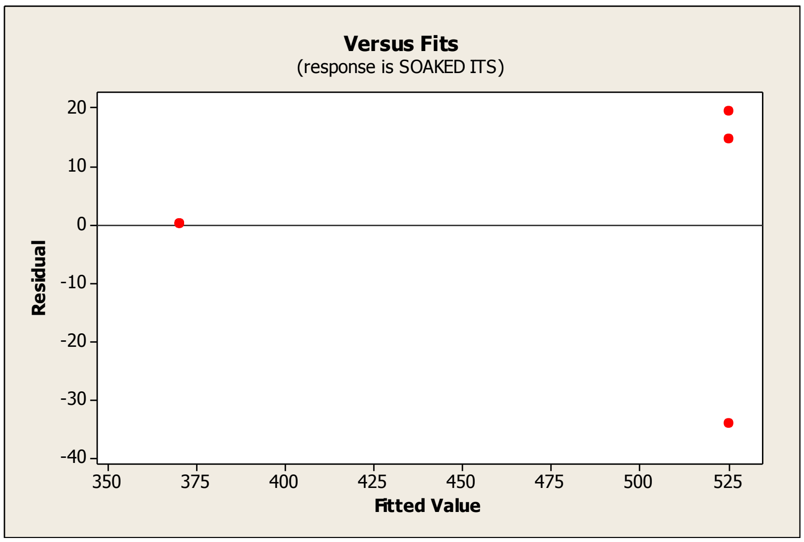


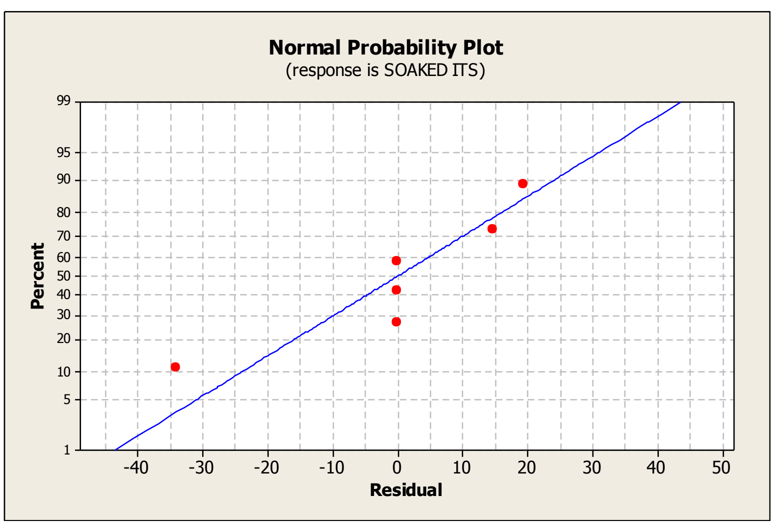
Minitab ANOVA printout: soaked ITS for marl soil.

One-way ANOVA: SOAKED ITS versus Treatment.

```
Source      DF     SS     MS      F      P
Treatment    1  36090  36090  82.18  0.001
Error        4   1757    439
Total        5  37847

S = 20.96   R-Sq = 95.36%   R-Sq(adj) = 94.20%.
```



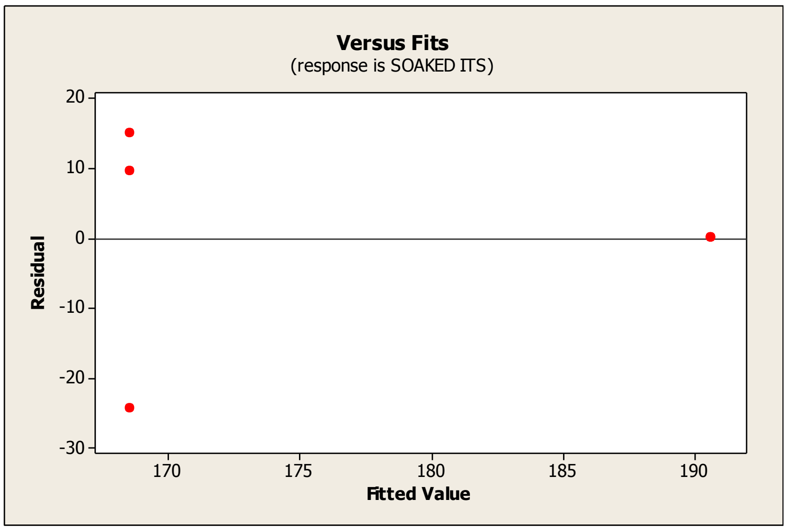


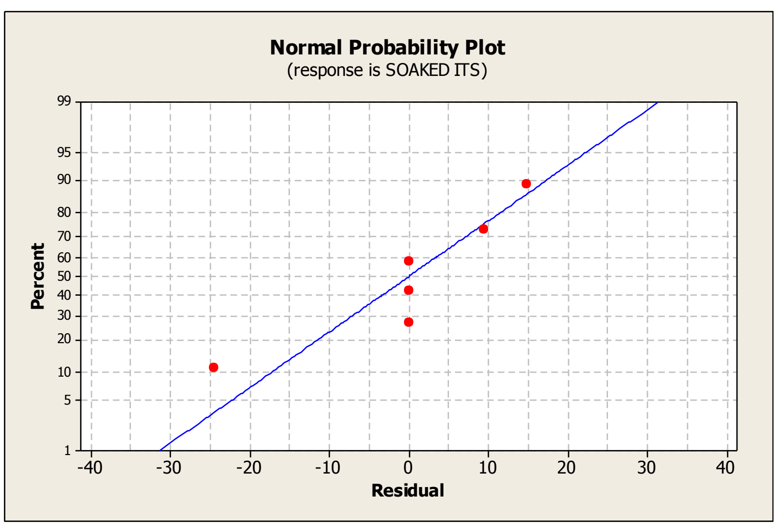
Minitab ANOVA printout: soaked ITS for sabkha soil.

One-way ANOVA: SOAKED ITS versus Treatment.

Source	DF	SS	MS	F	P
Treatment	1	735	735	3.23	0.147
Error	4	909	227		
Total	5	1644			

```
S = 15.07   R-Sq = 44.70%   R-Sq(adj) = 30.87%
```



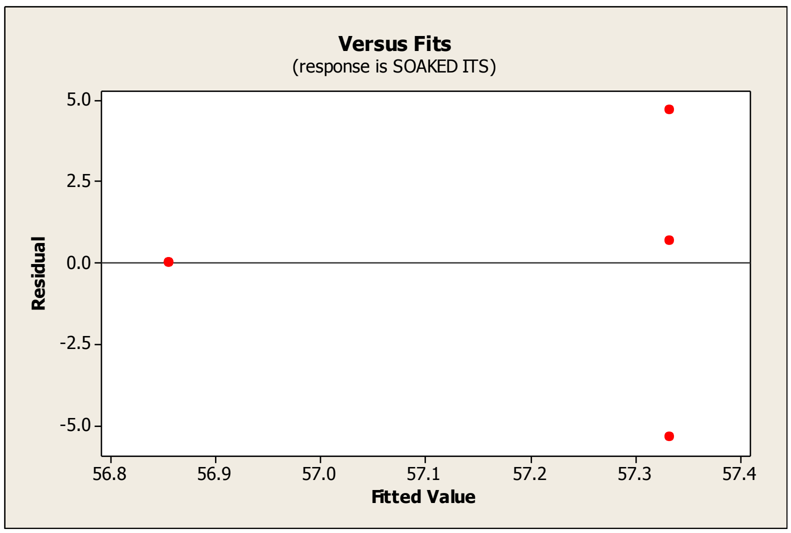


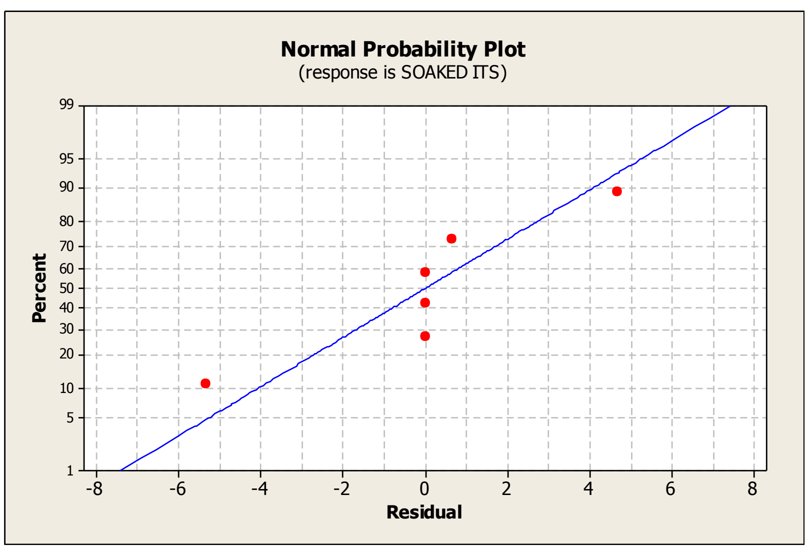
Minitab ANOVA printout: soaked ITS for sand soil.

One-way ANOVA: SOAKED ITS versus Treatment.

Source	DF	SS	MS	F	P
Treatment	1	0.3	0.3	0.03	0.877
Error	4	50.7	12.7		
Total	5	51.0			

S = 3.559   R-Sq = 0.67%   R-Sq(adj) = 0.00%



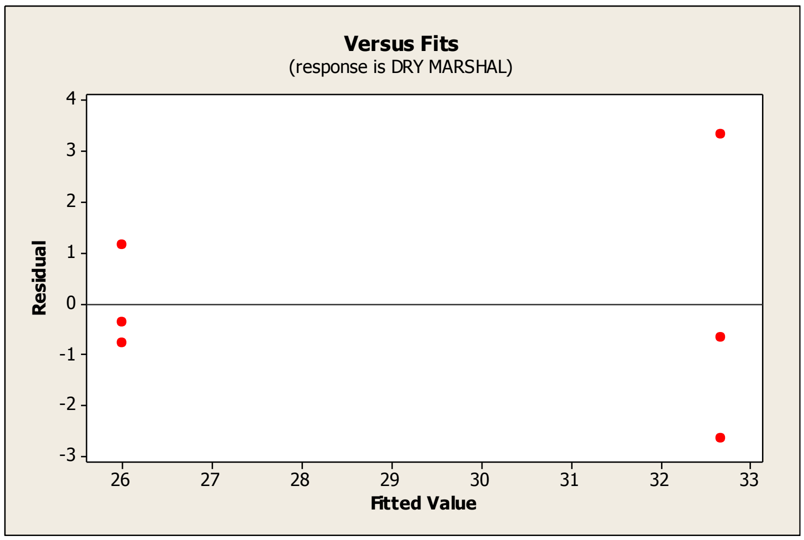


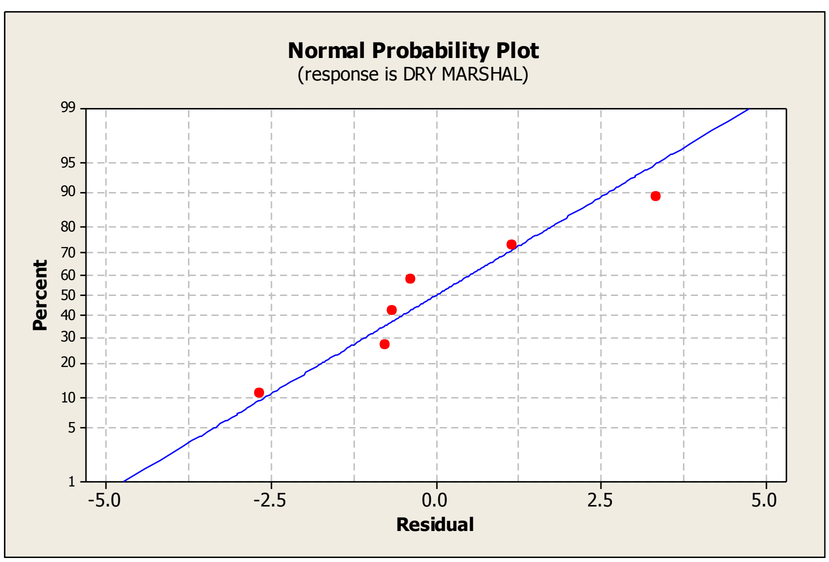
Minitab ANOVA printout: Dry stability for marl soil.

One-way ANOVA: Dry marshall versus treatment

```
Source      DF     SS     MS      F      P
Treatment    1  66.49  66.49  12.82  0.023
Error        4  20.75   5.19
Total        5  87.24

S = 2.278   R-Sq = 76.22%   R-Sq(adj) = 70.27%.
```



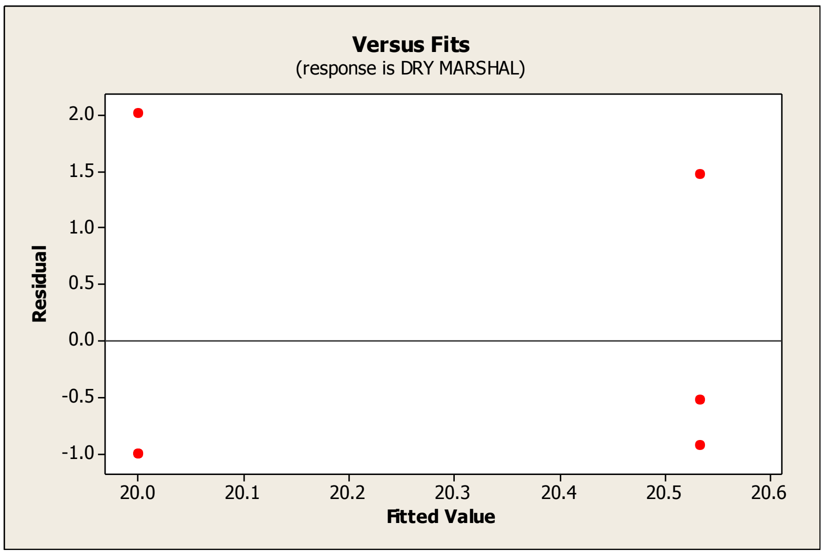


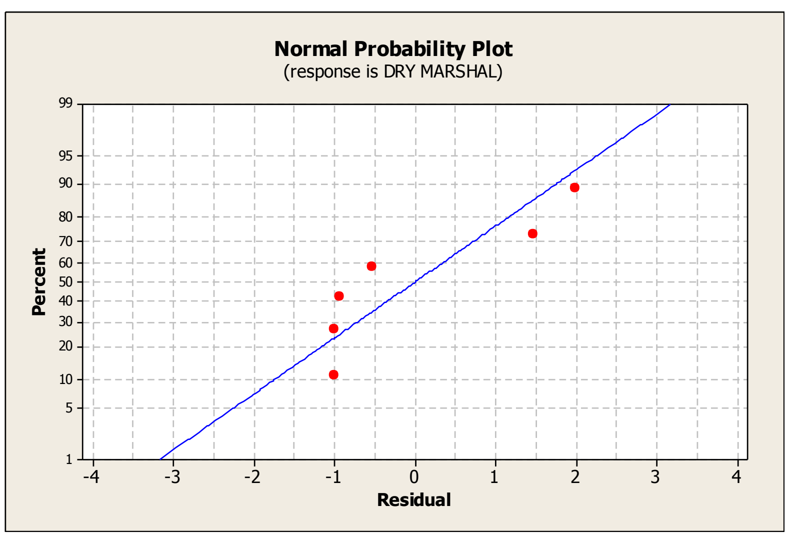
Minitab ANOVA printout: Dry stability for sabkha soil.

One-way ANOVA: Dry Marshall versus Treatment.

```
Source      DF    SS    MS     F      P
Treatment    1  0.43  0.43  0.18  0.691
Error        4  9.31  2.33
Total        5  9.73

S = 1.525   R-Sq = 4.38%   R-Sq(adj) = 0.00%
```



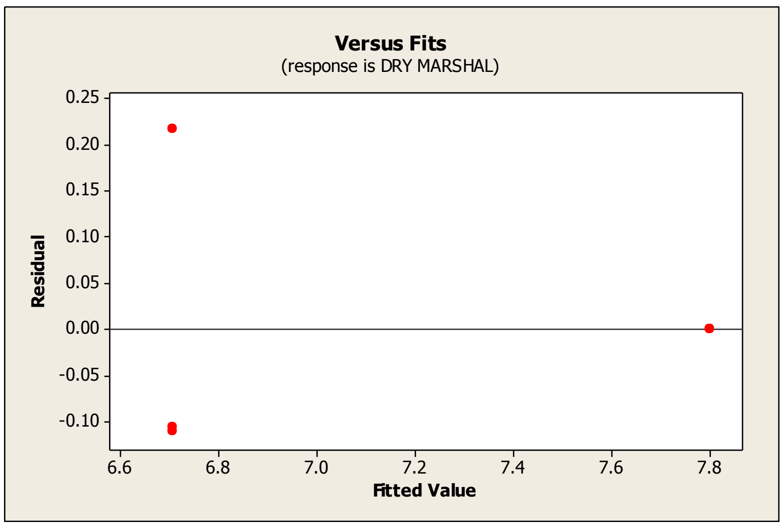


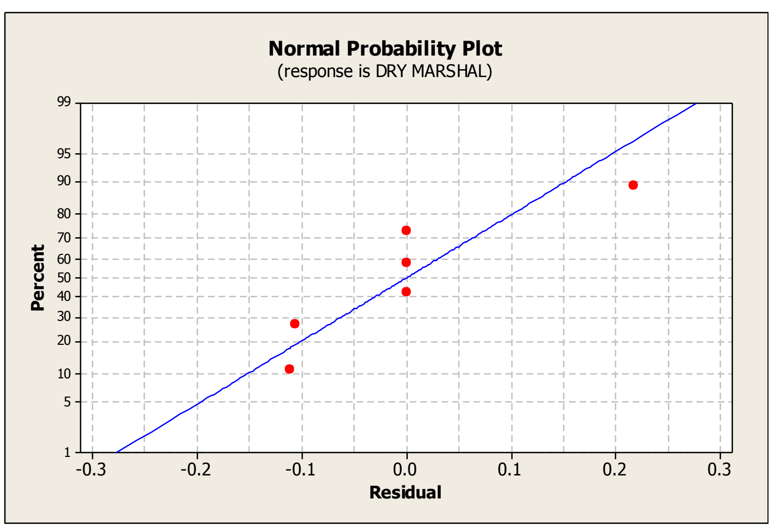
Minitab ANOVA printout: Dry stability for sand soil.

One-way ANOVA: Dry Marshall versus Treatment.

Source	DF	SS	MS	F	P
Treatment	1	1.7969	1.7969	101.94	0.001
Error	4	0.0705	0.0176		
Total	5	1.8674			

S = 0.1328   R-Sq = 96.22%   R-Sq(adj) = 95.28%



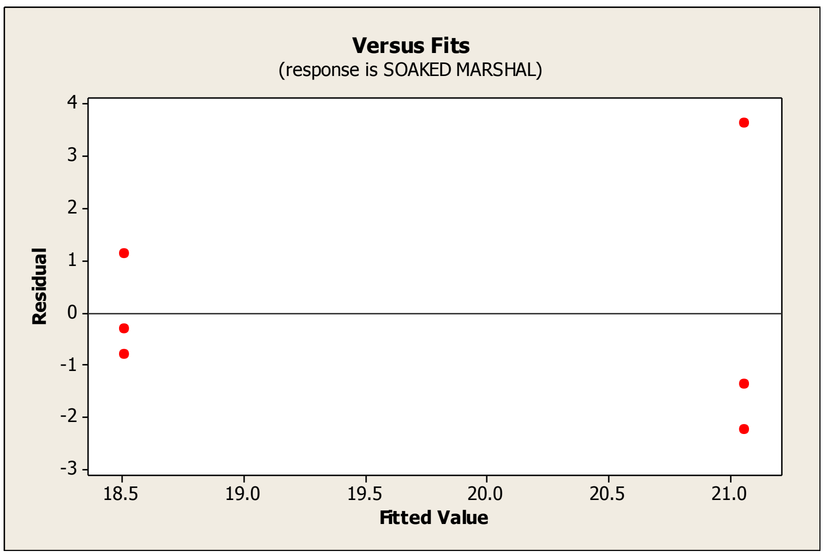


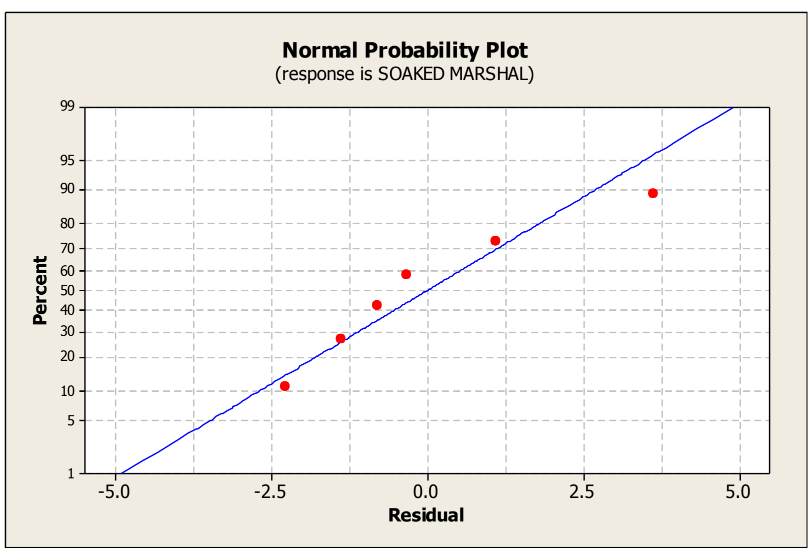
Minitab ANOVA printout: soaked stability for marl soil.

One-way ANOVA: SOAKED MARSHALL versus Treatment

Source	DF	SS	MS	F	P
Treatment	1	9.68	9.68	1.75	0.257
Error	4	22.14	5.53		
Total	5	31.82			

```
S = 2.353   R-Sq = 30.42%   R-Sq(adj) = 13.02%.
```



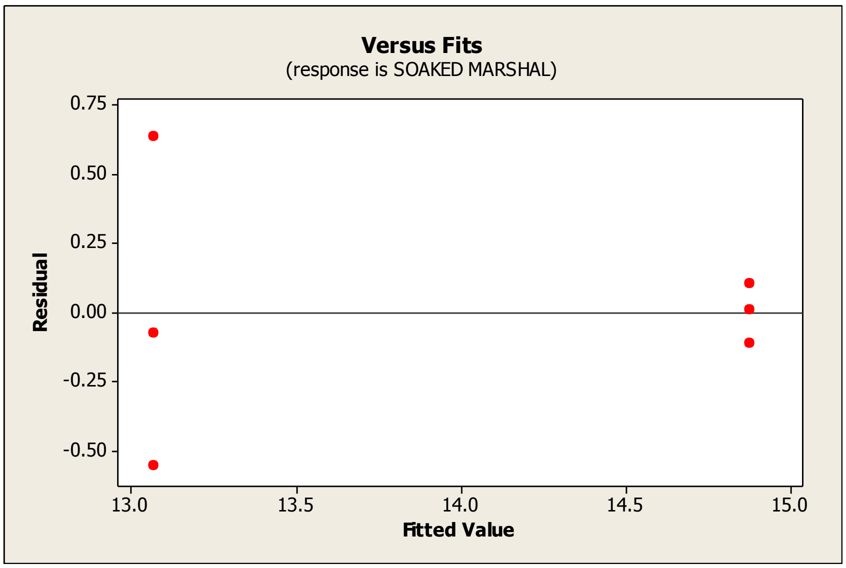


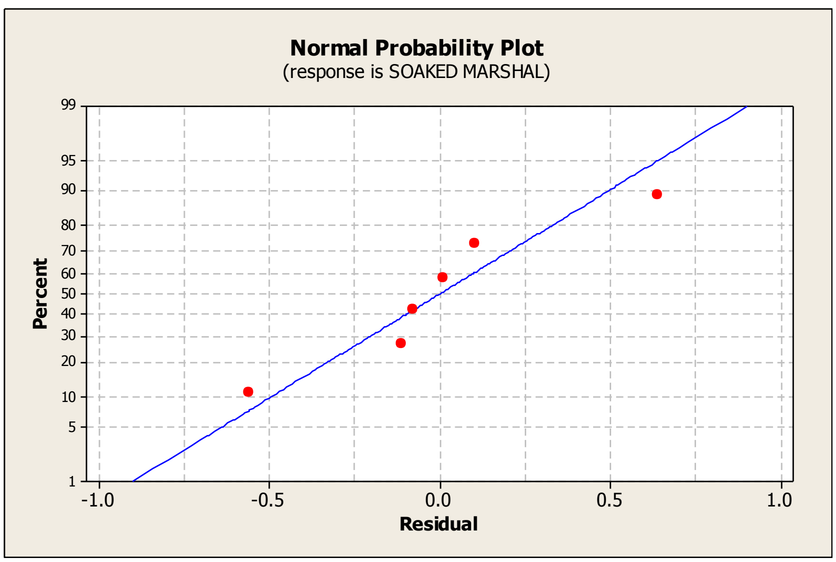
Minitab ANOVA printout: soaked stability for sabkha soil.

One-way ANOVA: SOAKED MARSHALL versus Treatment

Source	DF	SS	MS	F	P
Treatment	1	4.903	4.903	26.21	0.007
Error	4	0.748	0.187		
Total	5	5.651			

S = 0.4325   R-Sq = 86.76%   R-Sq(adj) = 83.45%.



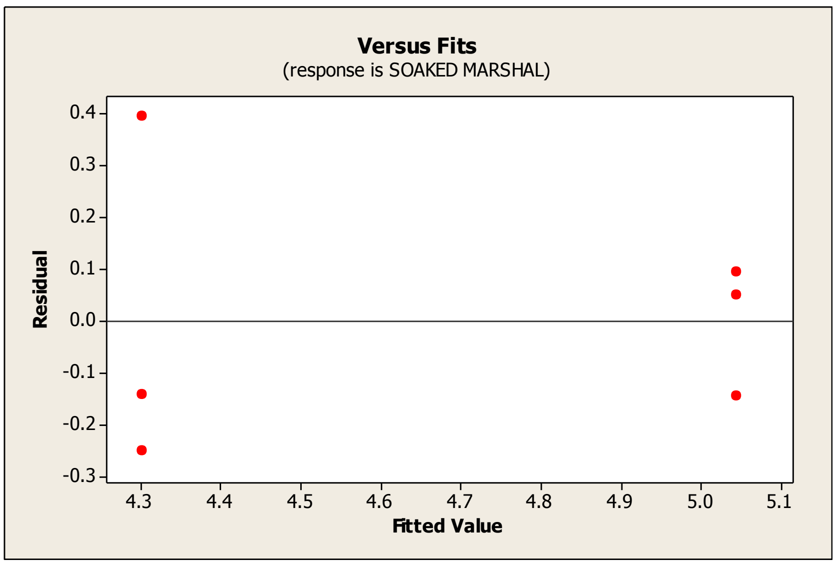


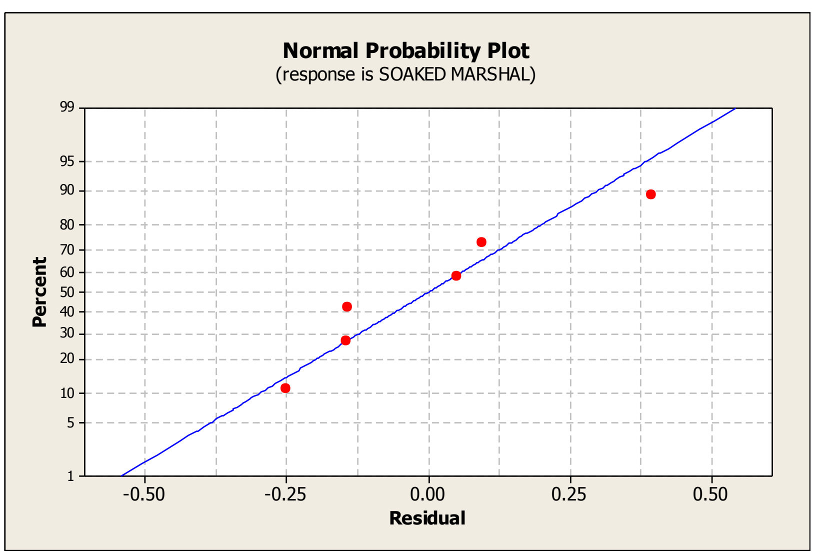
Minitab ANOVA printout: Soaked stability for sand soil.

One-way ANOVA: Soaked Marshall versus treatment.

```
Source      DF      SS      MS      F      P
Treatment    1  0.8294  0.8294  12.23  0.025
Error        4  0.2713  0.0678
Total        5  1.1007

S = 0.2604   R-Sq = 75.35%   R-Sq(adj) = 69.19%
```





Minitab ANOVA printout: Resilient modulus for marl soil.

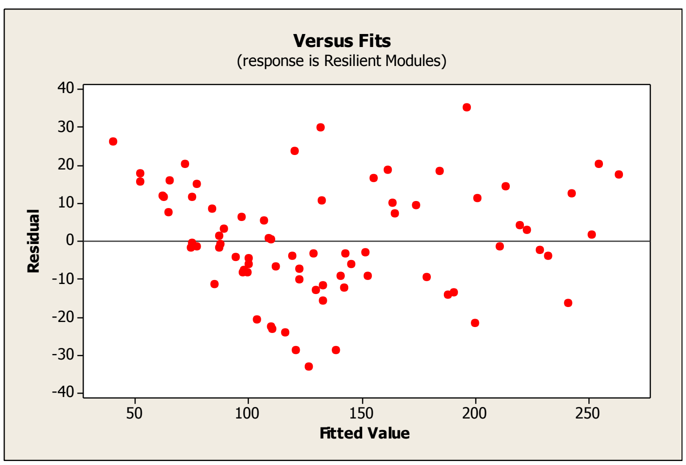
General Linear Model: Resilient M_R versus Temperature, Type of addi, ...

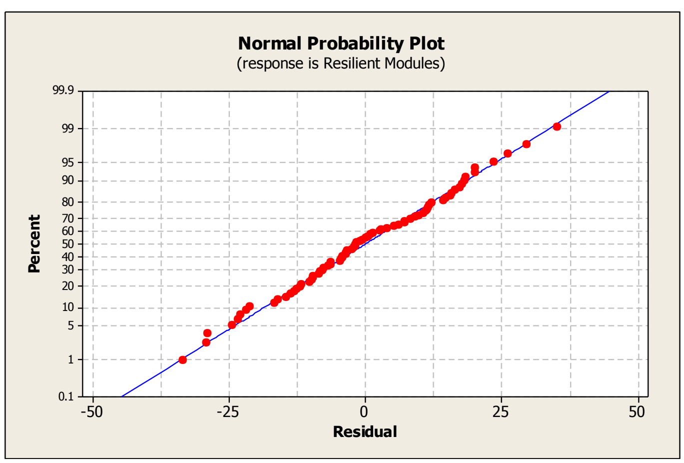
```
Factor              Type   Levels  Values
Temperature         fixed       2  1, 2
Type of additives   fixed       2  1, 2
Confining Pressure  fixed       5  1, 2, 3, 4, 5
Deviator stress     fixed       5  1, 2, 3, 4, 5


Analysis of Variance for Resilient Modules, using Adjusted SS for Tests

Source              DF  Seq SS  Adj SS  Adj MS       F      P
Temperature          1    2842    2842    2842   11.78  0.001
Type of additives    1    9800    9800    9800   40.61  0.000
Confining Pressure   4   80636   31620    7905   32.76  0.000
Deviator stress      4  140566  140566   35141  145.64  0.000
Error               65   15684   15684     241
Total               75  249528


S = 15.5338   R-Sq = 93.71%   R-Sq(adj) = 92.75%
```





Minitab ANOVA printout: Resilient modulus for sabkha soil.

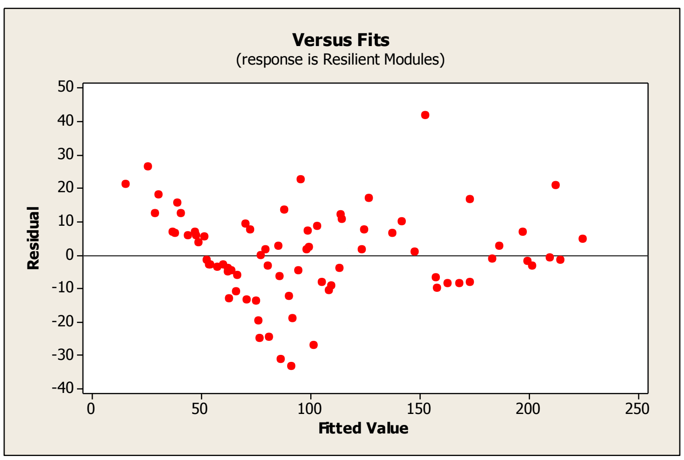
General Linear Model: Resilient M_R versus Temperature, Type of addi, ...

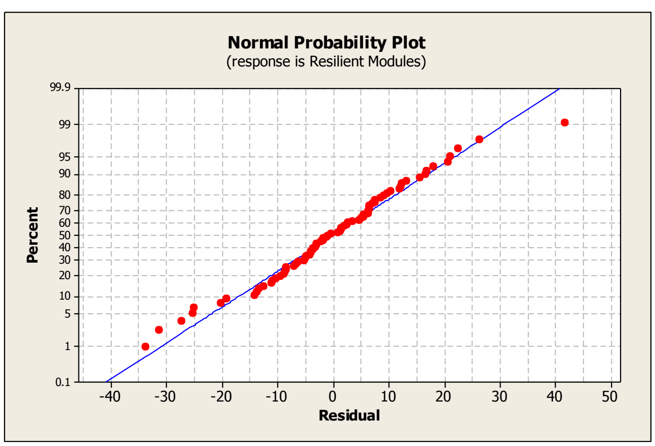
Factor	Type	Levels	Values
Temperature	fixed	2	1, 2
Type of additives	fixed	2	1, 2
Confining Pressure	fixed	5	1, 2, 3, 4, 5
Deviator stress	fixed	5	1, 2, 3, 4, 5

Analysis of Variance for Resilient Modules, using Adjusted SS for Tests

Source	DF	Seq SS	Adj SS	Adj MS	F	P
Temperature	1	2065	2065	2065	10.27	0.002
Type of additives	1	4317	4317	4317	21.46	0.000
Confining Pressure	4	75084	28344	7086	35.23	0.000
Deviator stress	4	127954	127954	31989	159.05	0.000
Error	65	13073	13073	201		
Total	75	222493				

S = 14.1816   R-Sq = 94.12%   R-Sq(adj) = 93.22%





Minitab ANOVA printout: Resilient modulus for sand soil.

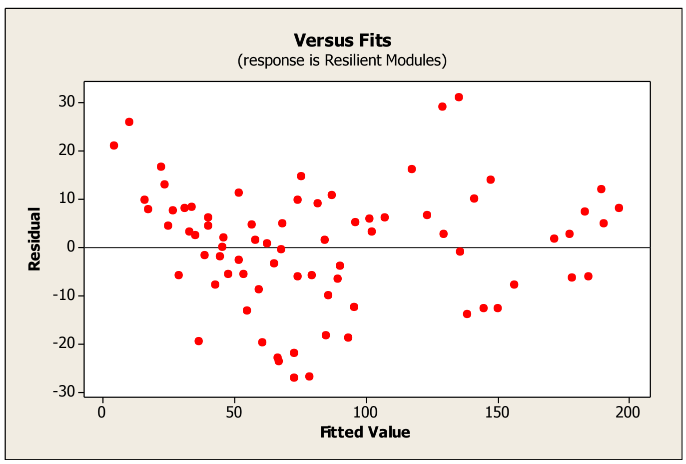
General Linear Model: Resilient M_R versus Temperature, Type of addi, ...

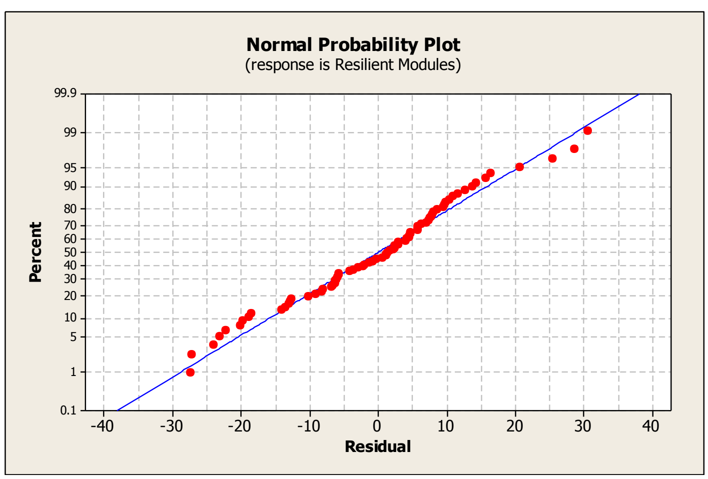
```
Factor               Type   Levels  Values
Temperature          fixed       2  1, 2
Type of additives    fixed       2  1, 2
Confining Pressure   fixed       5  1, 2, 3, 4, 5
Deviator stress      fixed       5  1, 2, 3, 4, 5


Analysis of Variance for Resilient Modules, using Adjusted SS for Tests

Source               DF  Seq SS  Adj SS  Adj MS       F      P
Temperature           1     709     709     709    4.03  0.049
Type of additives     1    2638    2638    2638   15.01  0.000
Confining Pressure    4   71775   30651    7663   43.61  0.000
Deviator stress       4  116031  116031   29008  165.10  0.000
Error                65   11420   11420     176
Total                75  202572


S = 13.2550   R-Sq = 94.36%   R-Sq(adj) = 93.50%
```





Minitab ANOVA printout: material property µ for investigated soil.

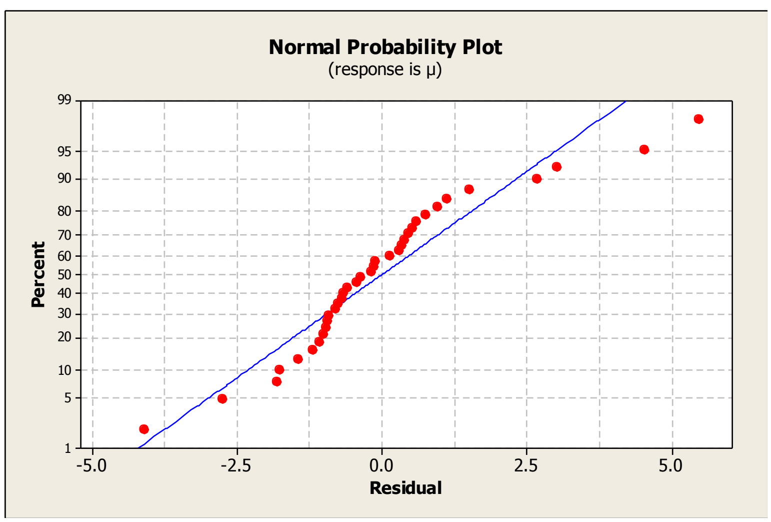
General Linear Model: µ versus Treatment; Temp.; Dev.

```
Factor      Type   Levels  Values
Treatment   fixed       2  1; 2
Temp.       fixed       2  22; 40
Dev.        fixed       5  40; 50; 60; 70; 80


Analysis of Variance for µ, using Adjusted SS for Tests

Source      DF   Seq SS   Adj SS  Adj MS      F      P
Treatment    1   11.664   11.664  11.664   2.95  0.096
Temp.        1   51.966   51.966  51.966  13.15  0.001
Dev.         4   26.391   26.391   6.598   1.67  0.184
Error       29  114.574  114.574   3.951
Total       35  204.594


S = 1.98767   R-Sq = 44.00%   R-Sq(adj) = 32.41%
```



Minitab ANOVA printout: material properties α and µ for investigated soil.

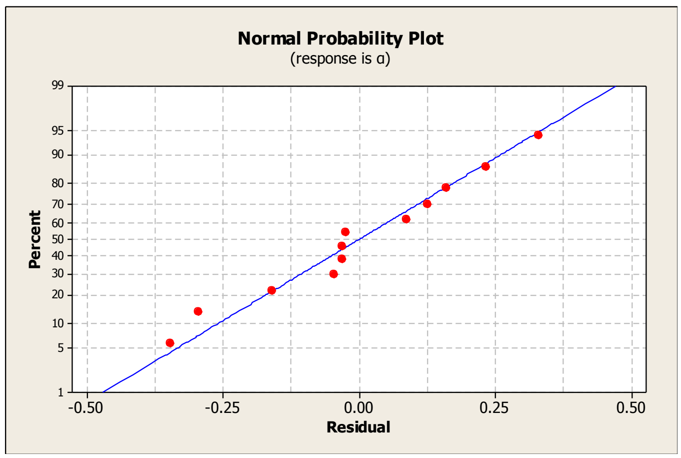
General Linear Model: µ versus Treatment; Temp.; Dev. For marl.

```
Factor      Type   Levels  Values
Treatment   fixed       2  1; 2
Temp.       fixed       2  1; 2
Dev.        fixed       3  60; 70; 80


Analysis of Variance for µ, using Adjusted SS for Tests
```

Source	DF	Seq SS	Adj SS	Adj MS	F	P
Treatment	1	0.37538	0.37538	0.37538	5.84	0.046
Temp.	1	0.38020	0.38020	0.38020	5.91	0.045
Dev.	2	0.72756	0.72756	0.36378	5.66	0.035
Error	7	0.45025	0.45025	0.06432		
Total	11	1.93339				

S = 0.253616   R-Sq = 76.71%   R-Sq(adj) = 63.40%



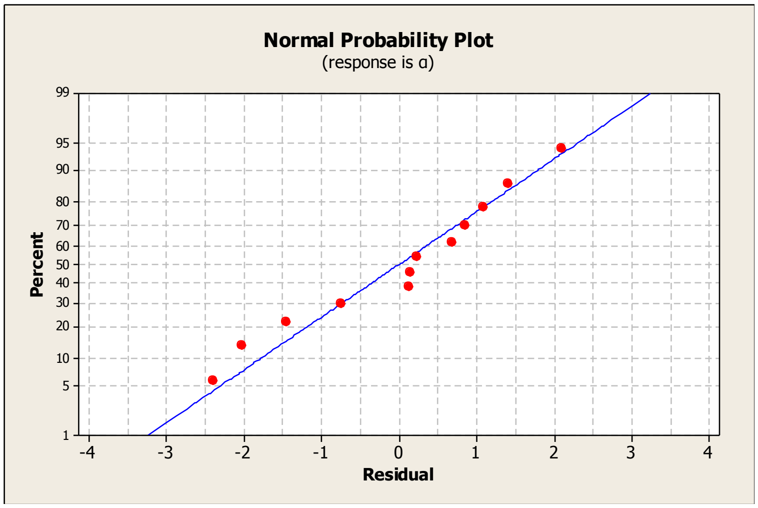
General Linear Model: α versus Treatment; Temp.; Dev.

Factor	Type	Levels	Values
Treatment	fixed	2	1; 2
Temp.	fixed	2	1; 2
Dev.	fixed	3	60; 70; 80

Analysis of Variance for α, using Adjusted SS for Tests

Source	DF	Seq SS	Adj SS	Adj MS	F	P
Treatment	1	0.000300	0.000300	0.000300	0.19	0.674
Temp.	1	0.000560	0.000560	0.000560	0.36	0.568
Dev.	2	0.012965	0.012965	0.006482	4.16	0.064
Error	7	0.010903	0.010903	0.001558		
Total	11	0.024728				

S = 0.0394655   R-Sq = 55.91%   R-Sq(adj) = 30.71%



General Linear Model: μ versus Treatment; Temp.; Deviator for dune sand.

Factor	Type	Levels	Values
Treatment	fixed	2	1; 2
Temp.	fixed	2	1; 2
Dev.	fixed	3	40; 50; 60

Analysis of Variance for μ , using Adjusted SS for Tests

Source	DF	Seq SS	Adj SS	Adj MS	F	P
Treatment	1	7.691	7.691	7.691	2.52	0.156
Temp.	1	79.121	79.121	79.121	25.94	0.001
Dev.	2	8.742	8.742	4.371	1.43	0.301
Error	7	21.350	21.350	3.050		
Total	11	116.904				

S = 1.74641   R-Sq = 81.74%   R-Sq(adj) = 71.30%

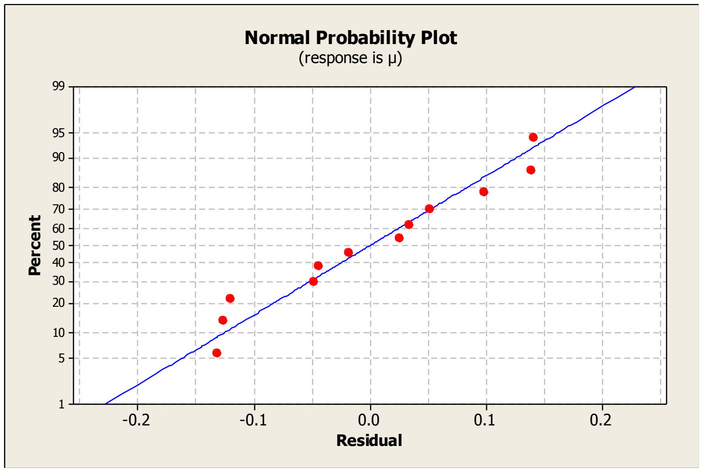
General Linear Model: α versus Treatment; Temp.; Dev.

Factor	Type	Levels	Values
Treatment	fixed	2	1; 2
Temp.	fixed	2	1; 2
Dev.	fixed	3	40; 50; 60

Analysis of Variance for α, using Adjusted SS for Tests

Source	DF	Seq SS	Adj SS	Adj MS	F	P
Treatment	1	0.01484	0.01484	0.01484	0.98	0.355
Temp.	1	0.13063	0.13063	0.13063	8.65	0.022
Dev.	2	0.28190	0.28190	0.14095	9.33	0.011
Error	7	0.10575	0.10575	0.01511		
Total	11	0.53311				

S = 0.122911   R-Sq = 80.16%   R-Sq(adj) = 68.83%



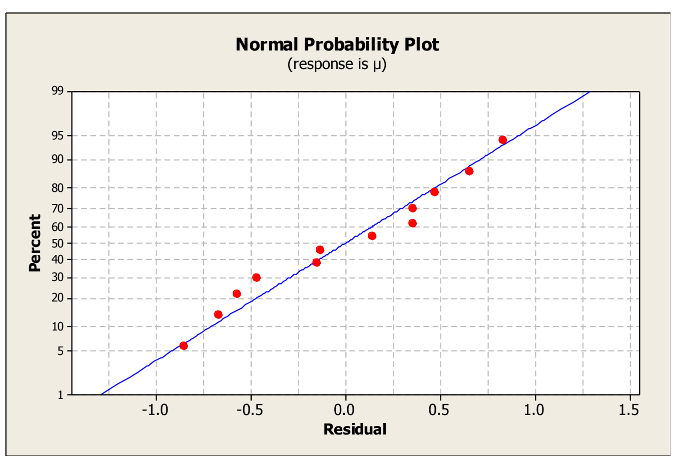
General Linear Model: μ versus Treatment; Temp.; Deviator stress for sabkha soil.

Factor	Type	Levels	Values
Treatment	fixed	2	1; 2
Temp.	fixed	2	1; 2
Dev.	fixed	3	40; 50; 60

Analysis of Variance for μ, using Adjusted SS for Tests

Source	DF	Seq SS	Adj SS	Adj MS	F	P
Treatment	1	6.3981	6.3981	6.3981	13.30	0.008
Temp.	1	8.8462	8.8462	8.8462	18.38	0.004
Dev.	2	2.3888	2.3888	1.1944	2.48	0.153
Error	7	3.3685	3.3685	0.4812		
Total	11	21.0016				

S = 0.693696   R-Sq = 83.96%   R-Sq(adj) = 74.80%



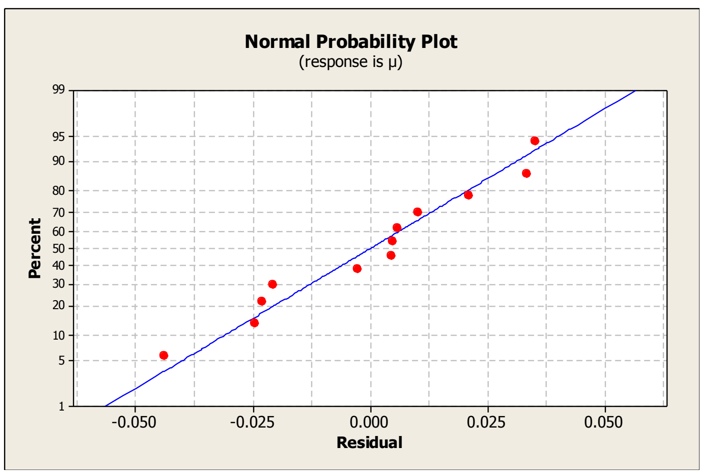
General Linear Model: α versus Treatment; Temp.; Dev.

Factor	Type	Levels	Values
Treatment	fixed	2	1; 2
Temp.	fixed	2	1; 2
Dev.	fixed	3	40; 50; 60

Analysis of Variance for α, using Adjusted SS for Tests

Source	DF	Seq SS	Adj SS	Adj MS	F	P
Treatment	1	0.0036750	0.0036750	0.0036750	3.98	0.086
Temp.	1	0.0003413	0.0003413	0.0003413	0.37	0.563
Dev.	2	0.0300162	0.0300162	0.0150081	16.23	0.002
Error	7	0.0064712	0.0064712	0.0009245		
Total	11	0.0405037				

S = 0.0304048   R-Sq = 84.02%   R-Sq(adj) = 74.89%



General Regression Analysis: μ versus T, σd, Treatment type for marl soil.

Regression Equation

Treatment type

FA $\mu = -1.72353 + 0.0197775\ T + 0.0298148\ \sigma d$

SFA $\mu = -2.07726 + 0.0197775\ T + 0.0298148\ \sigma d$

Coefficients

Term	Coef	SE Coef	T	P
Constant	-1.90039	0.647944	-2.93296	0.019
Treatment type				
FA	0.17687	0.069722	2.53673	0.035
T	0.01978	0.007747	2.55296	0.034
σd	0.02981	0.008539	3.49153	0.008

Summary of Model

S = 0.241524   R-Sq = 75.86%   R-Sq(adj) = 66.81%
PRESS = 1.17824 R-Sq(pred) = 39.06%

Analysis of Variance

Source	DF	Seq SS	Adj SS	Adj MS	F	P
Regression	3	1.46671	1.46671	0.488905	8.3811	0.0075017
Treatment type	1	0.37538	0.37538	0.375379	6.4350	0.0348881
T	1	0.38020	0.38020	0.380197	6.5176	0.0340178
σd	1	0.71114	0.71114	0.711138	12.1908	0.0081800
Error	8	0.46667	0.46667	0.058334		
Total	11	1.93339				

Fits and Diagnostics for Unusual Observations

No unusual observations

General Regression Analysis: μ versus T, σd, Treatment type for dune sand

Regression Equation

Treatment
type
FA   μ = -9.14798 + 0.285307 T + 0.10378 σd

SFA   μ = -10.7492 + 0.285307 T + 0.10378 σd

Coefficients

Term	Coef	SE Coef	T	P
Constant	-9.94858	3.35659	-2.96389	0.018
Treatment type				
FA	0.80060	0.47297	1.69268	0.129
T	0.28531	0.05255	5.42897	0.001
σd	0.10378	0.05793	1.79155	0.111

Summary of Model

S = 1.63843   R-Sq = 81.63%   R-Sq(adj) = 74.74%
PRESS = 52.2696 R-Sq(pred) = 55.29%

Analysis of Variance

Source	DF	Seq SS	Adj SS	Adj MS	F	P
Regression	3	95.429	95.4286	31.8095	11.8495	0.002588
Treatment type	1	7.691	7.6915	7.6915	2.8652	0.128972
T	1	79.121	79.1210	79.1210	29.4738	0.000624
σd	1	8.616	8.6162	8.6162	3.2097	0.110976
Error	8	21.476	21.4756	2.6845		
Total	11	116.904				

Fits and Diagnostics for Unusual Observations

No unusual observations

General Regression Analysis: μ versus T, σd, Treatment type

Regression Equation

Treatment type	
FA	$\mu = -2.49016 + 0.0953994\ T + 0.0539865\ \sigma d$
SFA	$\mu = -3.95054 + 0.0953994\ T + 0.0539865\ \sigma d$

Coefficients

Term	Coef	SE Coef	T	P
Constant	-3.22035	1.34059	-2.40219	0.043
Treatment type				
FA	0.73019	0.18890	3.86546	0.005
T	0.09540	0.02099	4.54520	0.002
σd	0.05399	0.02314	2.33348	0.048

Summary of Model

S = 0.654374   R-Sq = 83.69%   R-Sq(adj) = 77.57%
PRESS = 7.70937 R-Sq(pred) = 63.29%

Analysis of Variance

Source	DF	Seq SS	Adj SS	Adj MS	F	P
Regression	3	17.5760	17.5760	5.85867	13.6819	0.0016242
Treatment type	1	6.3981	6.3981	6.39815	14.9418	0.0047719
T	1	8.8462	8.8462	8.84622	20.6588	0.0018863
σd	1	2.3316	2.3316	2.33163	5.4451	0.0479008
Error	8	3.4256	3.4256	0.42821		
Total	11	21.0016				

Fits and Diagnostics for Unusual Observations

No unusual observations

General Regression Analysis: α versus T, σd, Treatment type for marl.

Regression Equation

Treatment type	
FA	$\alpha = 1.10005 + 0.000759259\ T - 0.004025\ \sigma d$
SFA	$\alpha = 1.11005 + 0.000759259\ T - 0.004025\ \sigma d$

Coefficients

Term	Coef	SE Coef	T	P
Constant	1.10505	0.0990560	11.1558	0.000
Treatment type				
FA	-0.00500	0.0106589	-0.4691	0.652
T	0.00076	0.0011843	0.6411	0.539
σd	-0.00402	0.0013054	-3.0832	0.015

Summary of Model

S = 0.0369236 R-Sq = 55.89% R-Sq(adj) = 39.35%
PRESS = 0.0271318 R-Sq(pred) = -9.72%

Analysis of Variance

Source	DF	Seq SS	Adj SS	Adj MS	F	P
Regression	3	0.0138208	0.0138208	0.0046069	3.37913	0.074820
Treatment type	1	0.0003000	0.0003000	0.0003000	0.22005	0.651523
T	1	0.0005603	0.0005603	0.0005603	0.41100	0.539392
σd	1	0.0129605	0.0129605	0.0129605	9.50633	0.015043
Error	8	0.0109068	0.0109068	0.0013634		
Total	11	0.0247277				

Fits and Diagnostics for Unusual Observations

No unusual observations

General Regression Analysis: α versus T, σd, Treatment type for dune sand.

Regression Equation

Treatment
type
FA α = 1.83245 - 0.0115926 T - 0.018075 σd

SFA α = 1.90279 - 0.0115926 T - 0.018075 σd

Coefficients

Term	Coef	SE Coef	T	P
Constant	1.86762	0.257393	7.25590	0.000
Treatment type				
FA	-0.03517	0.036269	-0.96960	0.361
T	-0.01159	0.004030	-2.87665	0.021
σd	-0.01808	0.004442	-4.06908	0.004

Summary of Model

S = 0.125640 R-Sq = 76.31% R-Sq(adj) = 67.43%
PRESS = 0.283998 R-Sq(pred) = 46.73%

Analysis of Variance

Source	DF	Seq SS	Adj SS	Adj MS	F	P

Regression	3	0.406830	0.406830	0.135610	8.5909	0.006974
Treatment type	1	0.014840	0.014840	0.014840	0.9401	0.360641
T	1	0.130625	0.130625	0.130625	8.2751	0.020617
σd	1	0.261365	0.261365	0.261365	16.5574	0.003588
Error	8	0.126283	0.126283	0.015785		
Total	11	0.533113				

Fits and Diagnostics for Unusual Observations

No unusual observations

General Regression Analysis: α versus T, σd, Treatment type for sabkha .

Regression Equation

Treatment
type

FA $\alpha = 1.06092 + 0.000592593\ T - 0.0061125\ \sigma d$

SFA $\alpha = 1.09592 + 0.000592593\ T - 0.0061125\ \sigma d$

Coefficients

Term	Coef	SE Coef	T	P
Constant	1.07842	0.0588309	18.3309	0.000
Treatment type				
FA	-0.01750	0.0082898	-2.1110	0.068
T	0.00059	0.0009211	0.6434	0.538
σd	-0.00611	0.0010153	-6.0204	0.000

Summary of Model

S = 0.0287167 R-Sq = 83.71% R-Sq(adj) = 77.60%
PRESS = 0.0160441 R-Sq(pred) = 60.39%

Analysis of Variance

Source	DF	Seq SS	Adj SS	Adj MS	F	P
Regression	3	0.0339065	0.0339065	0.0113022	13.7054	0.001615
Treatment type	1	0.0036750	0.0036750	0.0036750	4.4564	0.067765
T	1	0.0003413	0.0003413	0.0003413	0.4139	0.537991
σd	1	0.0298901	0.0298901	0.0298901	36.2458	0.000316
Error	8	0.0065972	0.0065972	0.0008247		
Total	11	0.0405037				

Fits and Diagnostics for Unusual Observations

Obs	α	Fit	SE Fit	Residual	St Resid	
10	0.696	0.742208	0.0175853	-0.0462083	-2.03538	R

R denotes an observation with a large standardized residual.

Regression model: τ versus treatment type, material type, C, ϕ for the investigated soil.

The regression equation is:

τ = 14.9 - 0.364 treatment type + 0.315 material type + 1.00 C - 0.0636 ϕ

Predictor	Coef	SE Coef	T	P
Constant	14.8900	0.5828	25.55	0.025
treatment type	-0.3643	0.1465	-2.49	0.243
material type	0.31490	0.07266	4.33	0.144
C	1.00112	0.00088	1133.11	0.001
ϕ	-0.06357	0.02509	-2.53	0.239

S = 0.135358   R-Sq = 100.0%   R-Sq(adj) = 100.0%

Analysis of Variance

Source	DF	SS	MS	F	P
Regression	4	44418	11105	606080.44	0.001
Residual Error	1	0	0		
Total	5	44418			

Source	DF	Seq SS
treatment type	1	14004
material type	1	3517
C	1	26897
ϕ	1	0

Unusual Observations

Obs	treatment type	τ	Fit	SE Fit	Residual	St Resid
1	1.00	135.475	135.479	0.135	-0.004	-1.00 X
4	2.00	299.160	299.164	0.135	-0.004	-1.00 X

X denotes an observation whose X value gives it large leverage.

Regression model: μ versus Treatment type, Material type, T, σ_d for the investigated soil.

The regression equation is:

μ = - 0.49 + 0.671 Treatment type - 1.14 Material type + 0.133 T - 0.0148 σ_d

Predictor	Coef	SE Coef	T	P
Constant	-0.494	3.234	-0.15	0.880
Treatment type	0.6705	0.5405	1.24	0.224
Material type	-1.1384	0.6672	-1.71	0.098
T	0.13349	0.03707	3.60	0.001
σ_d	-0.01478	0.03538	-0.42	0.679

S = 2.00157   R-Sq = 39.3%   R-Sq(adj) = 31.5%

Analysis of Variance

Source	DF	SS	MS	F	P
Regression	4	80.400	20.100	5.02	0.003
Residual Error	31	124.195	4.006		
Total	35	204.594			

Source	DF	Seq SS
Treatment type	1	16.071
Material type	1	11.664
T	1	51.966
σd	1	0.699

Unusual Observations

Obs	Treatment type	μ	Fit	SE Fit	Residual	St Resid
13	2.00	9.106	4.162	0.590	4.945	2.59R
19	2.00	8.908	3.023	0.590	5.885	3.08R

R denotes an observation with a large standardized residual.

Regression model: α versus Treatment type, Material type, T, σ_d

The regression equation is:

α = 0.981 - 0.0447 Treatment type + 0.0384 Material type - 0.00341 T - 0.00169 σ_d

Predictor	Coef	SE Coef	T	P
Constant	0.9809	0.2768	3.54	0.001
Treatment type	-0.04474	0.04627	-0.97	0.341
Material type	0.03844	0.05711	0.67	0.506
T	-0.003414	0.003173	-1.08	0.290
σd	-0.001691	0.003029	-0.56	0.581

S = 0.171332   R-Sq = 7.6%   R-Sq(adj) = 0.0%

Analysis of Variance

Source	DF	SS	MS	F	P
Regression	4	0.07502	0.01875	0.64	0.639
Residual Error	31	0.91000	0.02935		
Total	35	0.98502			

Source	DF	Seq SS
Treatment type	1	0.01859
Material type	1	0.01330
T	1	0.03398
σd	1	0.00915

Unusual Observations

Obs	Treatment type	α	Fit	SE Fit	Residual	St Resid
19	2.00	0.1990	0.7304	0.0505	-0.5314	-3.25R

R denotes an observation with a large standardized residual.

Regression model: M_R versus M. type, Treatment type, ...

The regression equation is:

M_R = 89.0 - 19.6 M. type - 7.24 Treatment type - 0.376 Tem.
+ 0.101 Confining Pressure + 4.80 Deviator stress

Predictor	Coef	SE Coef	T	P
Constant	88.983	3.152	28.23	0.000
M. type	-19.5989	0.6573	-29.82	0.000
Treatment type	-7.239	1.446	-5.00	0.000
Tem.	-0.37649	0.05967	-6.31	0.000
Confining Pressure	0.1015	0.1122	0.90	0.366
Deviator stress	4.79821	0.05820	82.44	0.000

S = 14.0371   R-Sq = 94.1%   R-Sq(adj) = 94.0%

Analysis of Variance

Source	DF	SS	MS	F	P
Regression	5	2114820	422964	2146.58	0.000
Residual Error	678	133594	197		
Total	683	2248414			

Source	DF	Seq SS
M. type	1	175158
Treatment type	1	172875
Tem.	1	10773
Confining Pressure	1	416798
Deviator stress	1	1339218

Unusual Observations

Obs	M. type	Mr	Fit	SE Fit	Residual	St Resid
57	1.00	231.550	198.824	1.681	32.726	2.35R
60	1.00	216.150	184.808	1.362	31.342	2.24R
67	1.00	179.185	144.110	1.064	35.075	2.51R
69	1.00	173.745	144.110	1.064	29.635	2.12R
73	1.00	237.320	192.092	1.186	45.228	3.23R
74	1.00	226.220	192.092	1.186	34.128	2.44R
75	1.00	231.880	192.092	1.186	39.788	2.84R
79	1.00	280.440	240.074	1.536	40.366	2.89R
80	1.00	269.340	240.074	1.536	29.266	2.10R
81	1.00	275.000	240.074	1.536	34.926	2.50R
91	1.00	167.085	120.626	1.369	46.459	3.33R
92	1.00	155.985	120.626	1.369	35.359	2.53R
93	1.00	161.645	120.626	1.369	41.019	2.94R
97	1.00	177.535	144.617	1.265	32.918	2.35R
103	1.00	231.325	192.599	1.249	38.726	2.77R
105	1.00	225.885	192.599	1.249	33.286	2.38R

106	1.00	214.825	185.823	1.251	29.002	2.07R
109	1.00	286.490	240.582	1.483	45.908	3.29R
110	1.00	275.390	240.582	1.483	34.808	2.49R
111	1.00	281.050	240.582	1.483	40.468	2.90R
398	2.00	142.950	171.986	1.204	-29.036	-2.08R
410	2.00	95.045	124.511	0.837	-29.466	-2.10R
416	2.00	138.825	172.493	0.987	-33.668	-2.40R
417	2.00	143.825	172.493	0.987	-28.668	-2.05R
425	2.00	184.035	213.698	1.385	-29.663	-2.12R
428	2.00	48.790	77.036	1.373	-28.246	-2.02R
434	2.00	72.220	101.027	1.200	-28.807	-2.06R
440	2.00	92.790	125.018	1.080	-32.228	-2.30R
505	3.00	72.660	111.644	1.446	-38.984	-2.79R
506	3.00	72.440	111.644	1.446	-39.204	-2.81R
507	3.00	78.100	111.644	1.446	-33.544	-2.40R
526	3.00	130.290	98.135	1.057	32.155	2.30R
589	3.00	60.220	31.924	1.418	28.296	2.03R

R denotes an observation with a large standardized residual.

Vitae

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